

# DESIGN EXPO 2012

## *Geotechnical Issues*

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# DESIGN EXPO 2012

- Dilatancy and its potential impact on driven pile capacity
  - Duval County – Relaxation factors up to 0.8 (BOR/EOD = 0.8)
  - The use of CPT-U for preliminary investigations
- Punching shear analysis for driven pile foundations
  - Pile groups on thin bearing layers



# DILATANCY

- Focus on granular soil
- Dilation is the observed tendency of dense granular material to dilate (expand in volume) as it is sheared.
- In densely packed arrangements interlocking prevents the grains from moving around each other and they are forced to either shear or “roll” over each other.

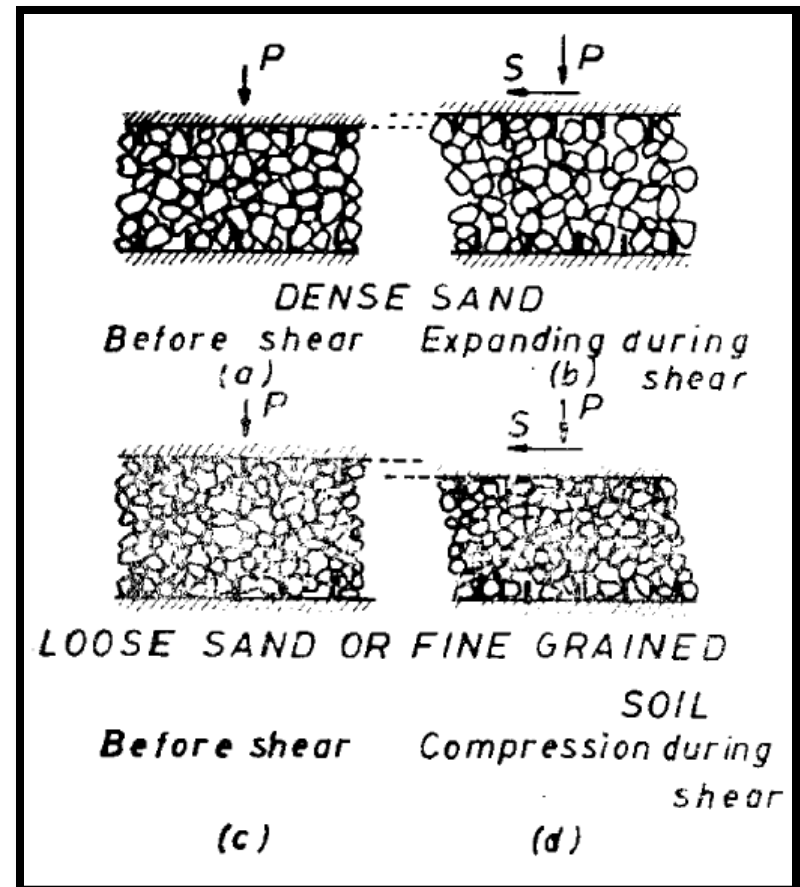
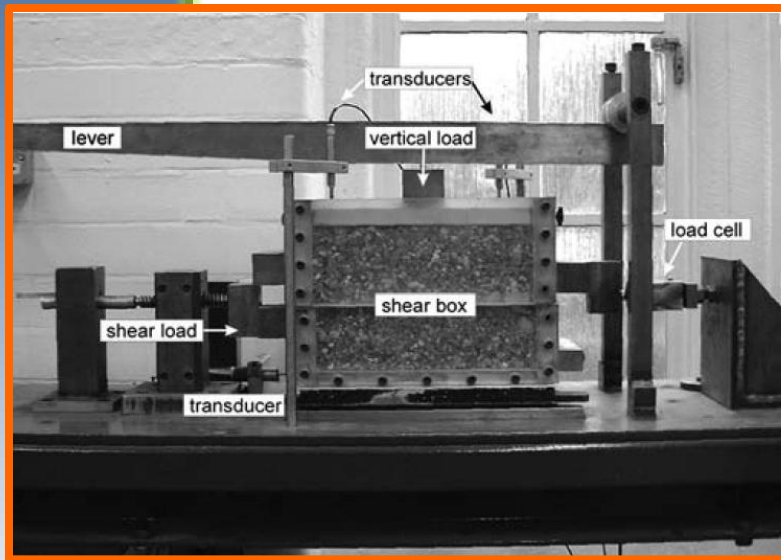


# DILATANCY

- As suggested by Schofield (2000) dilatancy may have been recognized as being part of the shearing mechanism of granular materials as far back as 1726 by Couplet
- “It is a remarkable fact that a dense sand, when compressed in one direction actually increases in volume.” (Lambe and Whitman 1969)



# DILATANCY





# DILATANCY

- Coulomb's description of shear strength does not explicitly state the influence of dilation

$$\tau = c' + \sigma' \tan \phi'$$

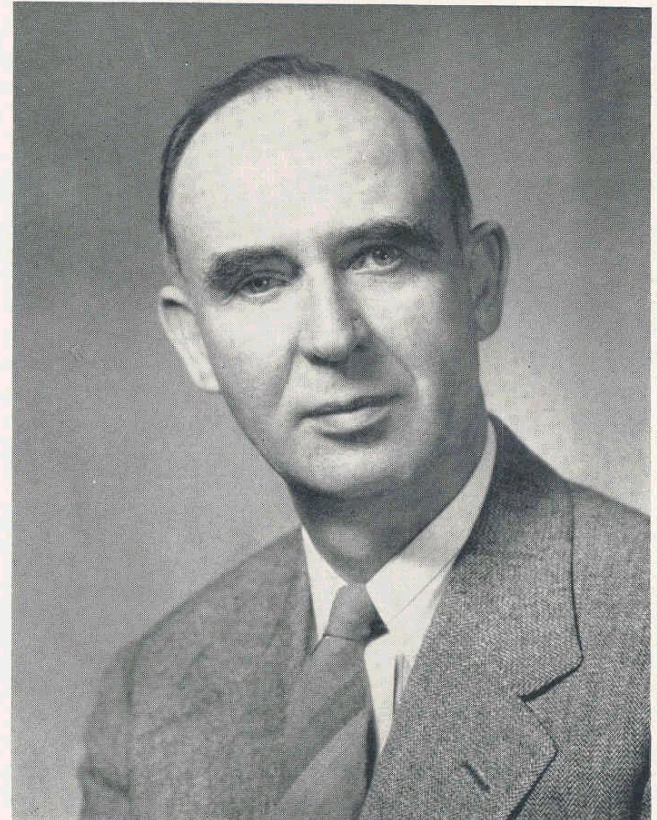




# DILATANCY

- Taylor (1948) proposed that there was a relationship between peak shear strength and dilation, and added an “interlocking” force to the frictional force

- $\tau \delta x = \sigma \delta y + \mu \sigma \delta x$

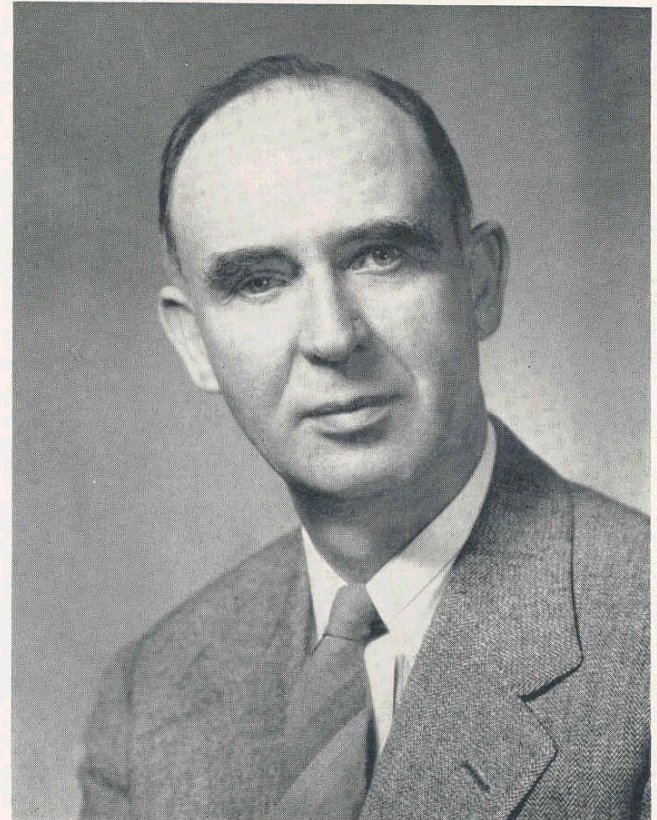


Donald W. Taylor



# DILATANCY

- “The angle of internal friction, in spite of its name, does not depend solely on internal friction, since a portion of the shearing stress on a plane of failure is utilized in overcoming interlocking.”

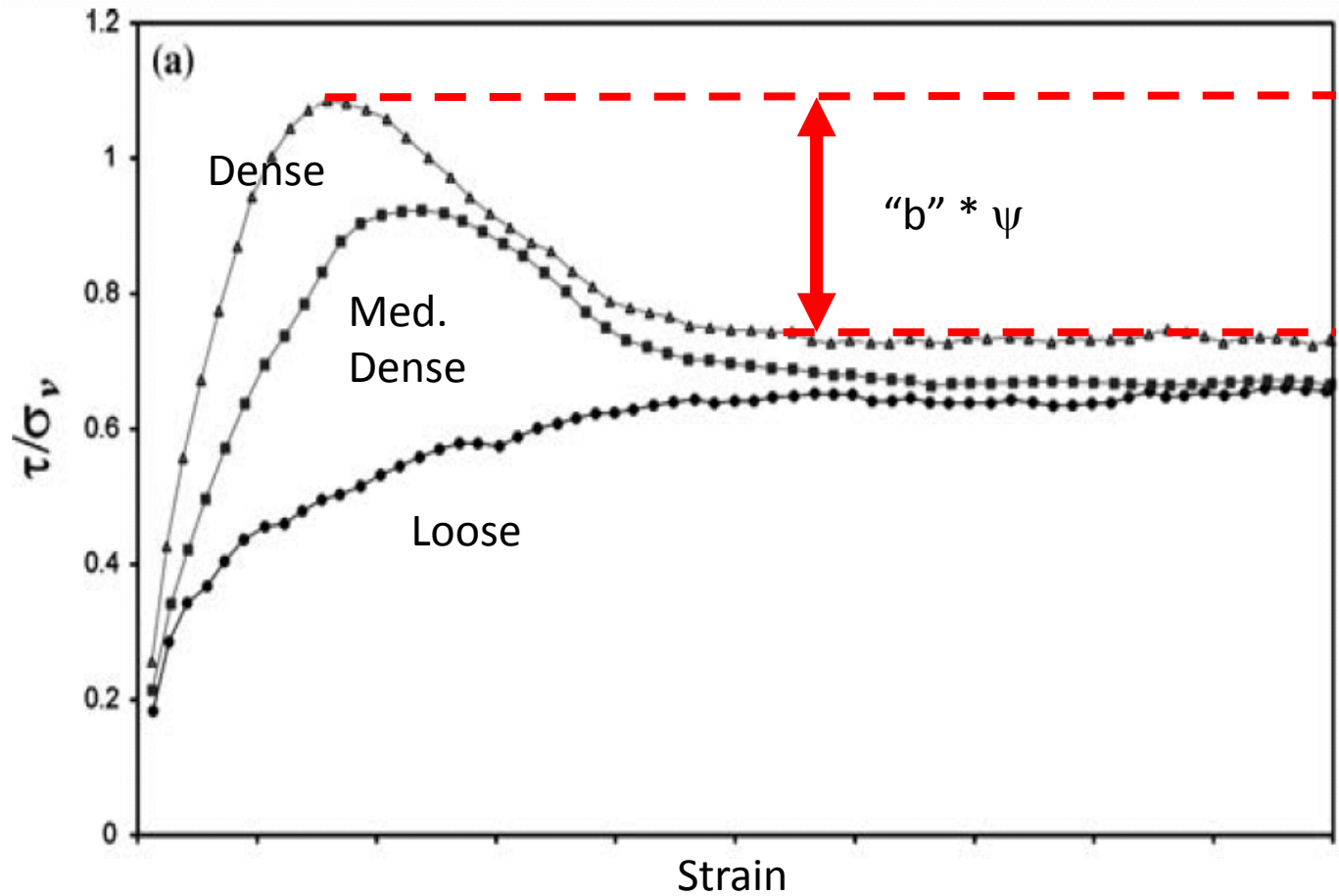


Donald W. Taylor

Fundamentals of Soil Mechanics, 1948



# DILATANCY



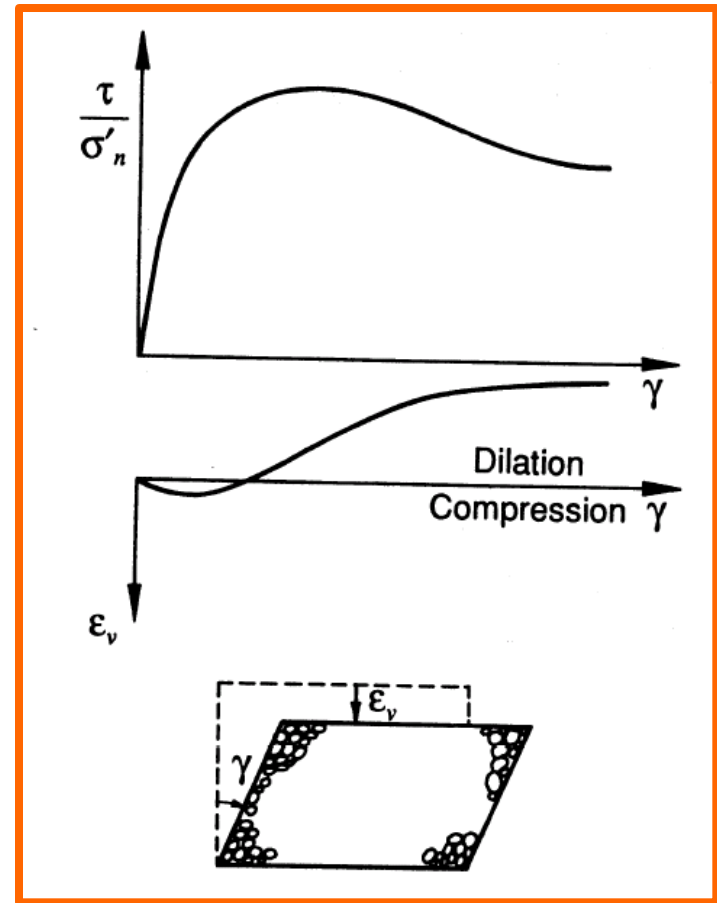
$$\tan \phi'_p = \tan(\phi'_{cv} + \psi)$$

(General form)



# DILATANCY

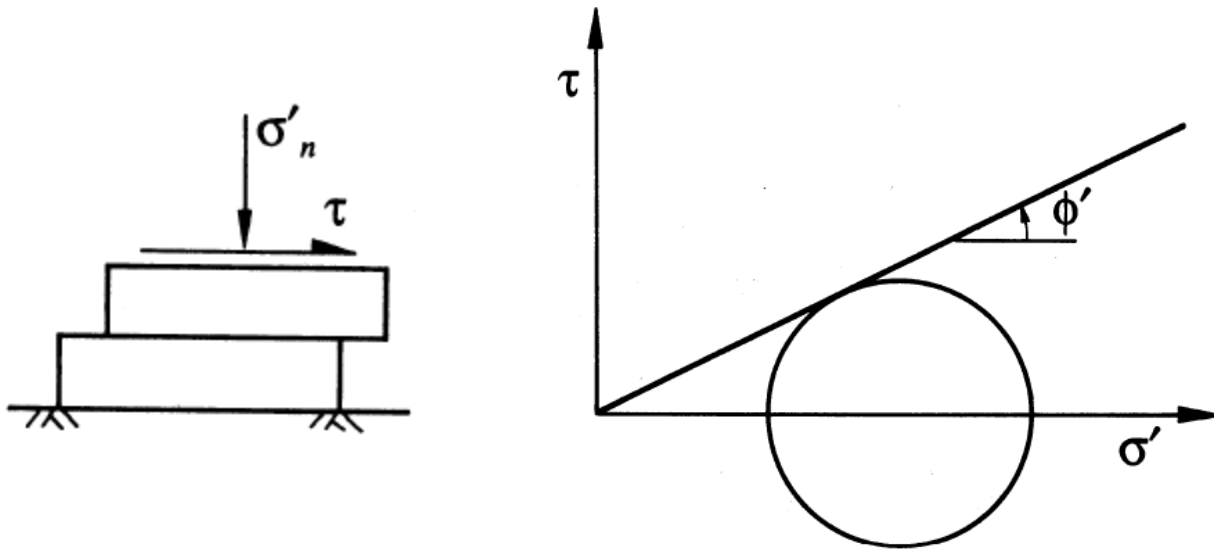
- Dense sands tend to dilate
- Initial compression followed by an expansion in volume
- Denser samples dilate faster
- When the test is performed under large pressure  $\phi'_p$  approaches  $\phi'_{cv}$



After Houlsby



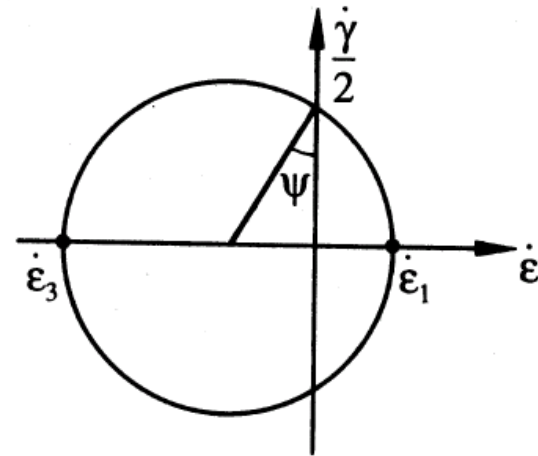
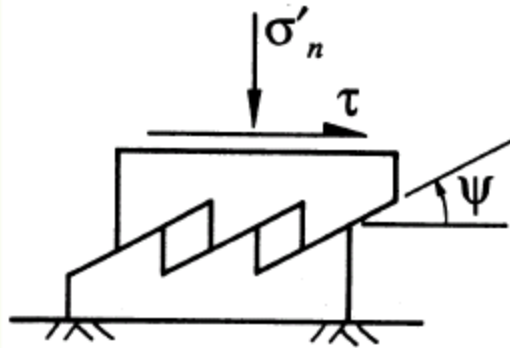
# DILATANCY



$\tan \phi'$  is the ratio of shear stress to normal stress



# DILATANCY



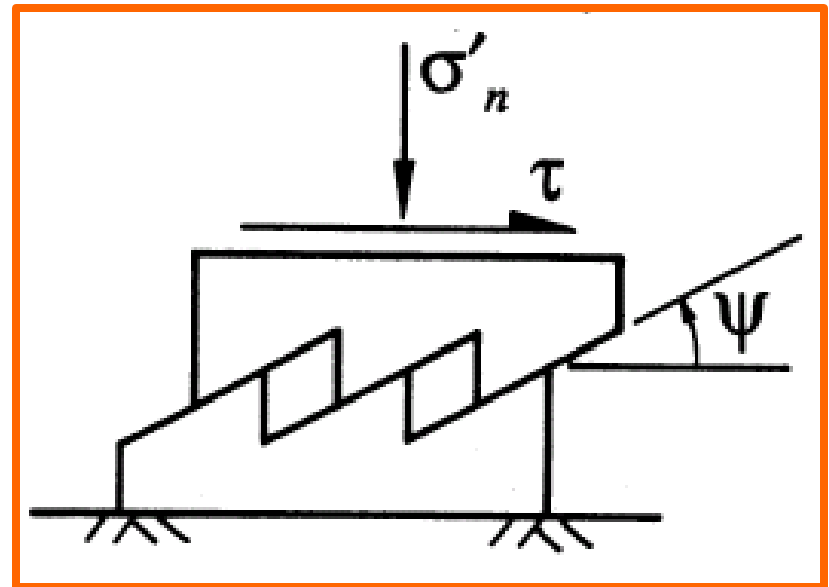
Mohr's Circle for Strain Rate

$\tan \psi$  is the ratio between a volumetric strain rate and a shear strain rate



# DILATANCY

- Saw tooth blocks model
- Friction  $\phi'_{cv}$  between blocks



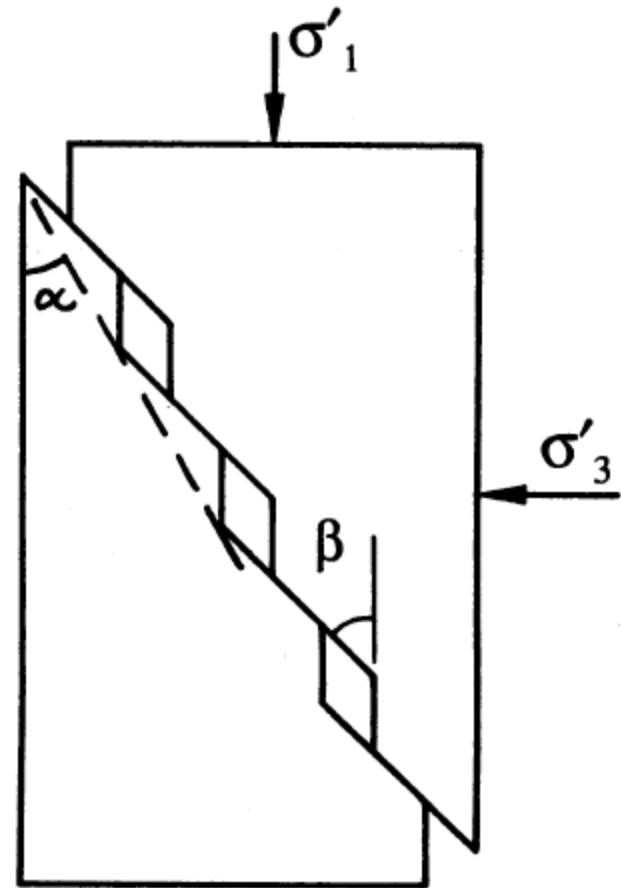
Flow rule

$$\tau/\sigma'_n = \tan \phi' = \tan(\phi'_{cv} + \psi)$$



# DILATANCY

- Other models
- Taylor
- Schofield and Wroth
- Rowe
- Bolton



Rowe's model for a stress-dilatancy flow rule

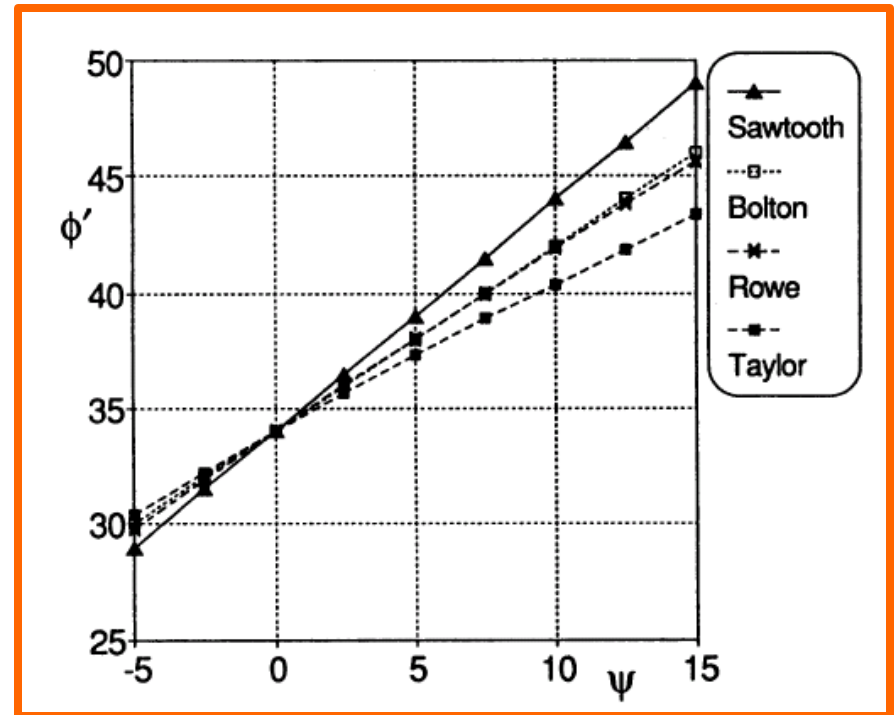


# DILATANCY

- Bolton
- Empirical stress-dilatancy flow rule based on direct shear tests

$$\phi'_p = \phi'_{cv} + 0.8\psi_{max}$$

Relationship between dilatancy and friction



After Houlsby



# DILATANCY

- Bolton

- For Plane Strain

$$\phi'_p = \phi'_{cv} + 0.8\psi_{max} = 5 I_R$$

- Relative dilatancy index  $I_R$

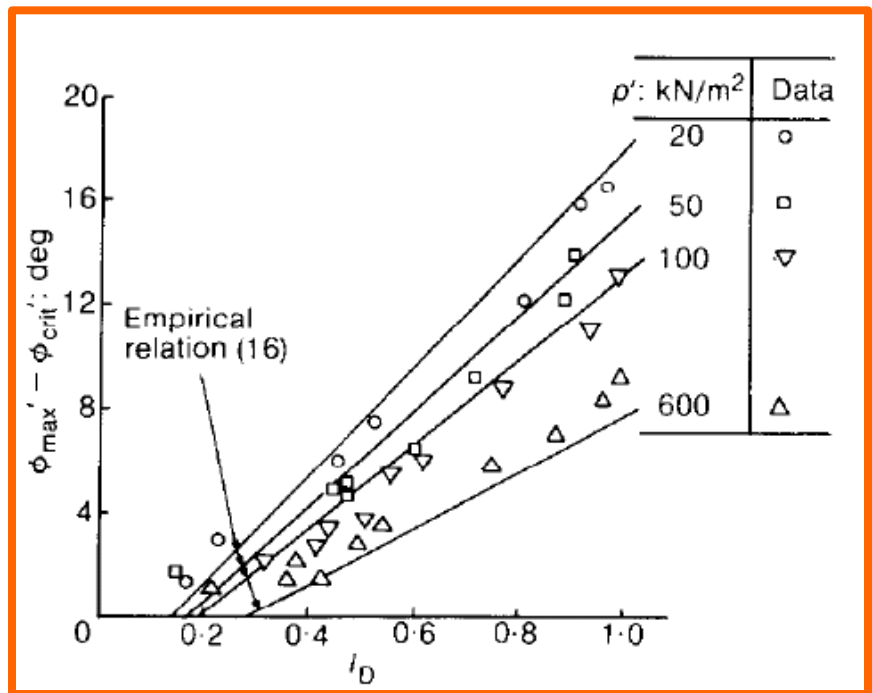
$$I_R = I_D (10 - \log p') - 1$$



# DILATANCY

- The angle of dilation tends to increase with density, that process is highly dependent on soil mineralogy

Dilatancy-density relationship

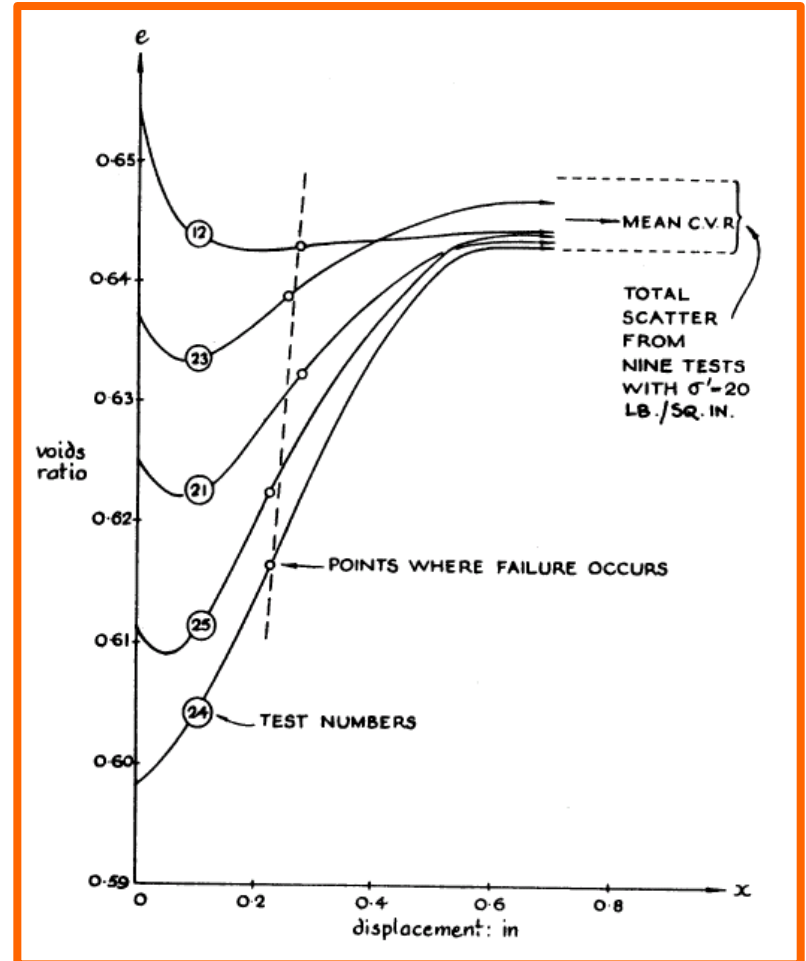


Berlin Sand (DeBeer, 1965)



# DILATANCY

- Samples dilate until they reach a constant volume void ratio regardless of their initial density

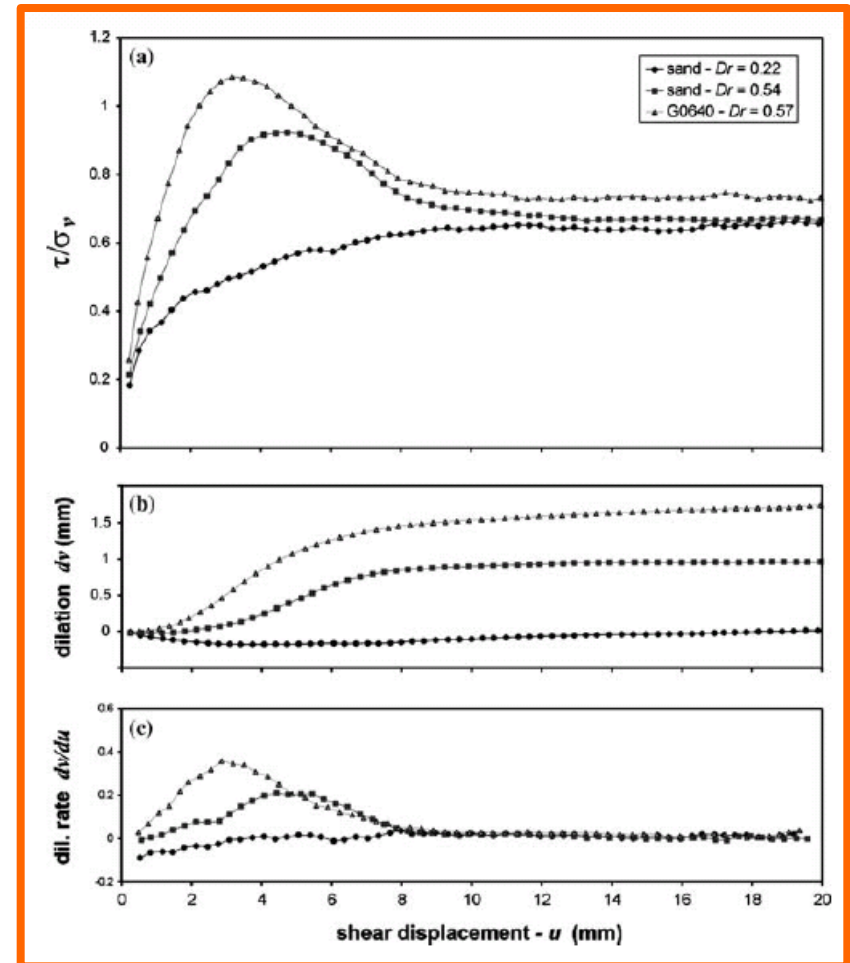




# DILATANCY

- The largest rate of dilation  $\tan \psi$  coincides with  $\phi'_{\text{peak}}$ .
- dilation rate approaches zero as  $\phi'$  approaches  $\phi'_{\text{cv}}$

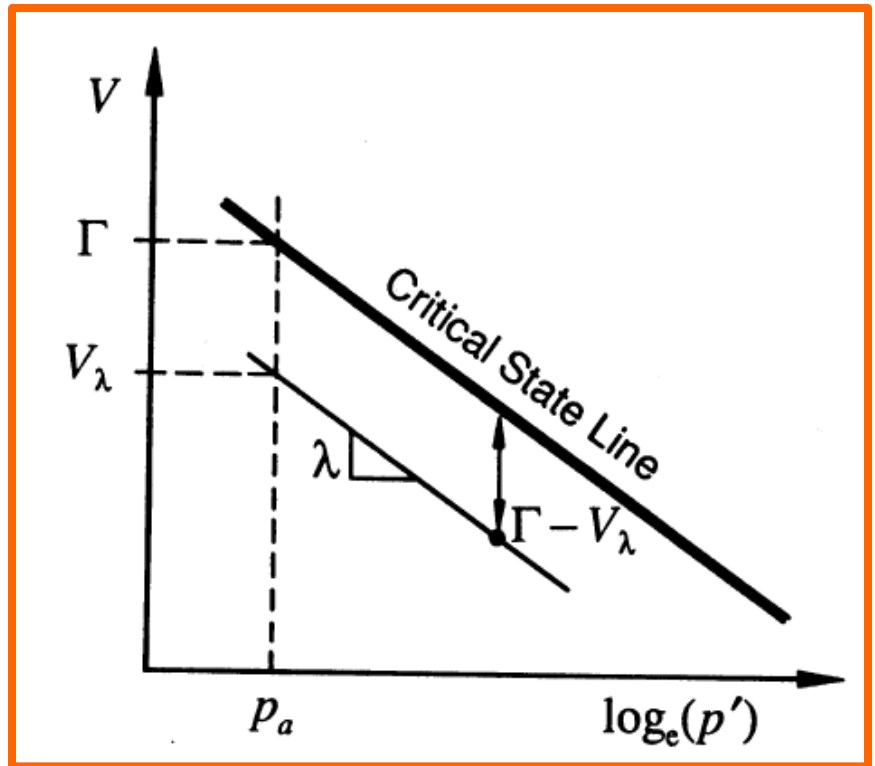
Simoni & Houlsby





# DILATANCY

- On the Critical State Line (CSL) the rate of dilation is zero
- Dilation will only occur a certain “distance” from the CSL in terms of stress/strain

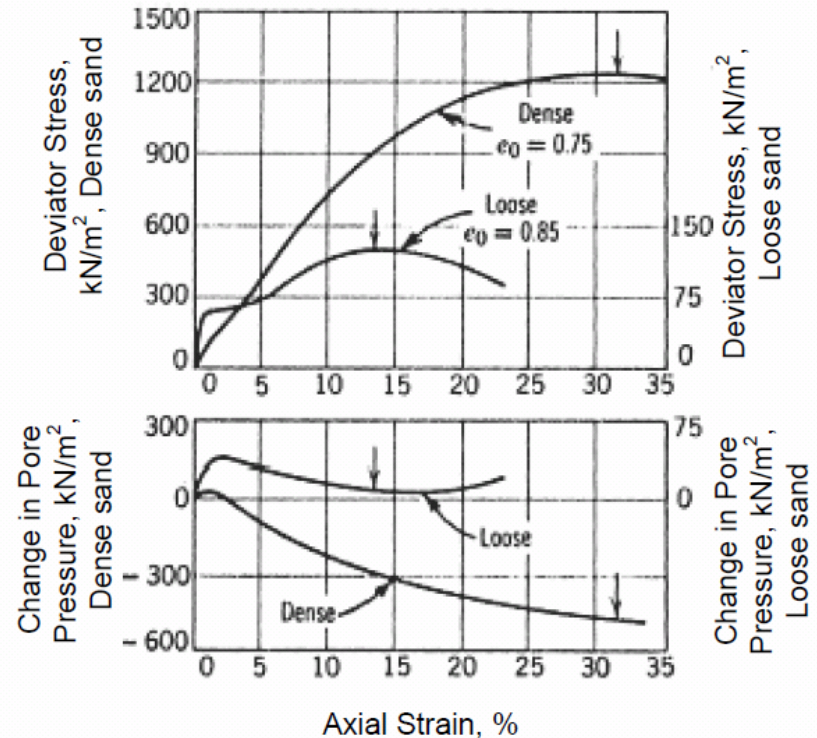




# DILATANCY

- Dilatant soil is capable of generating large negative pore water pressures, having an overall effect of a **temporary increase in effective stress**

$$\sigma'_v = \sigma_T - U$$



After Leonards



# DILATANCY

- At rest

$$\sigma'_{v t=0} = \sigma_T - U_o$$

- During dilation

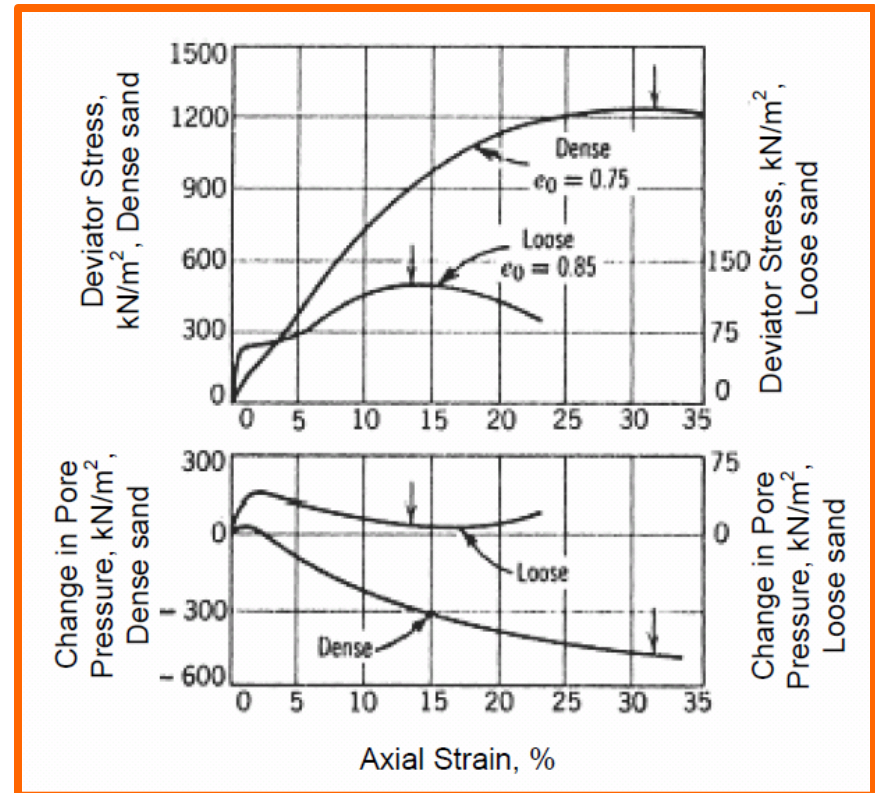
$$\sigma'_{v t=1} = \sigma_T - (-U_1)$$

$$\sigma'_{v t=1} > \sigma'_{v t=0}$$

- After time “t”

$U_1$  will revert to  $U_o$

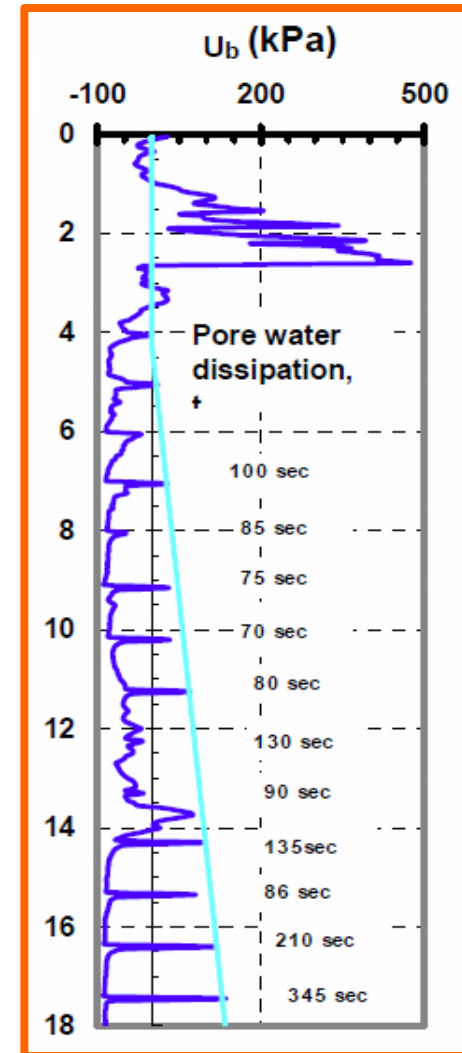
$$\sigma'_{v t=2} \leq \sigma'_{v t=1}$$





# DILATANCY

- Seismic piezocone soundings in piedmont soils (ML and SM) indicate negative pore pressure generated from dilation is not a permanent condition, and “U” will return to a hydrostatic stress level after a period of time

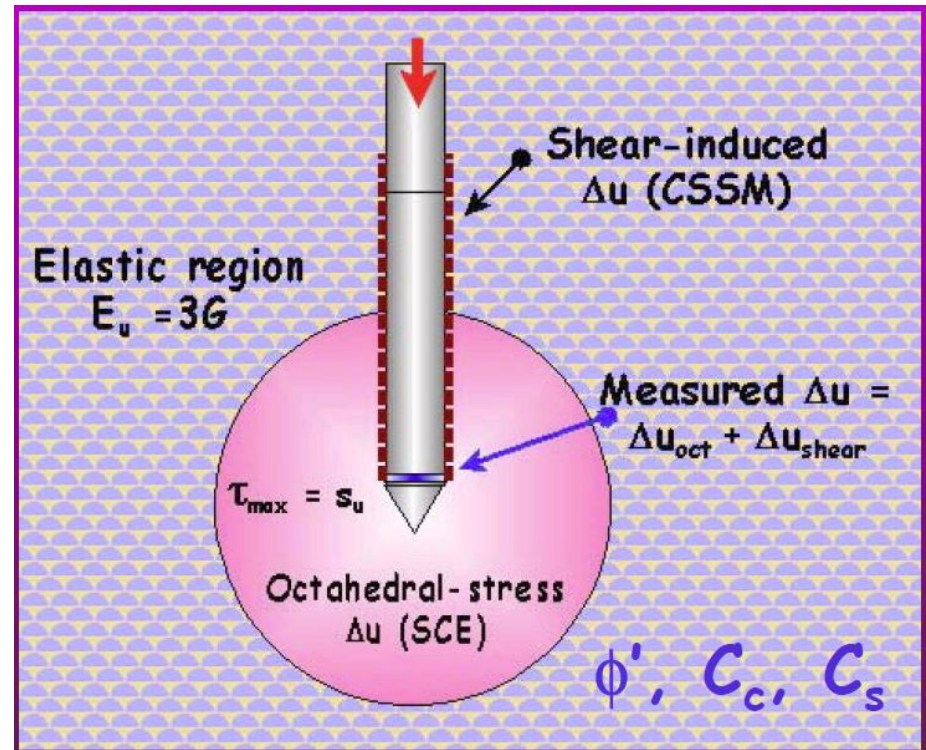


After Mayne



# DILATANCY

- Mayne's model for the CPTU combines cylindrical cavity expansion theory with Critical State Soil Mechanics





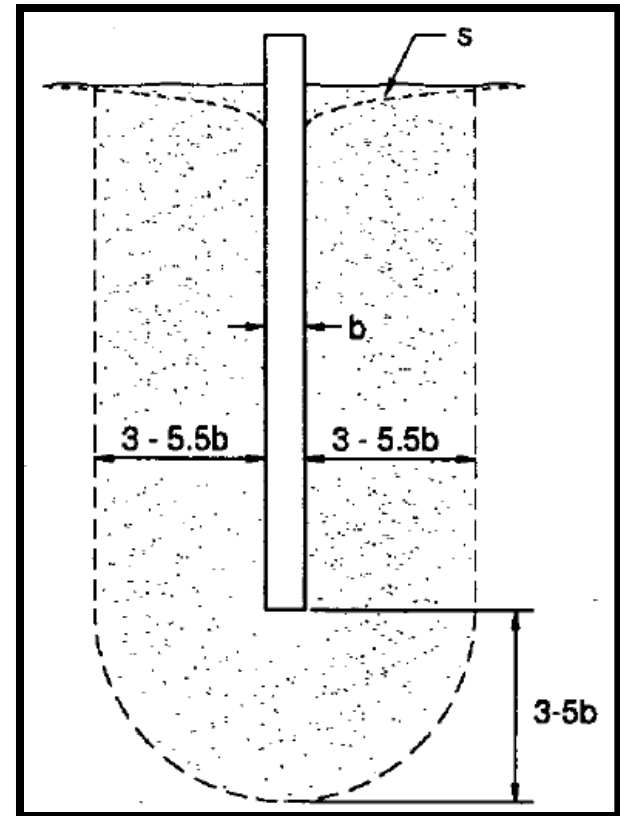
# DILATANCY





# DILATANCY

- Pile driving densifies the material and dilation is possible near the pile-soil interface

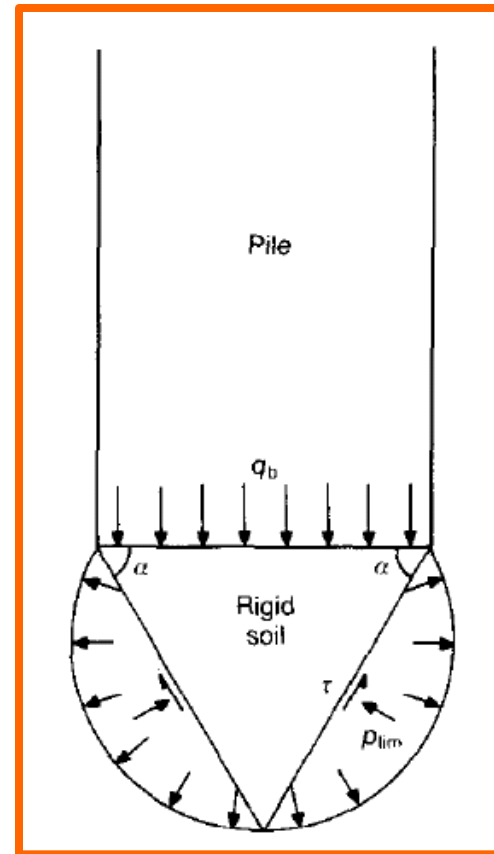


Zone of densification for granular soils due to pile driving. Broms, 1966



# DILATANCY

- Randolph proposes the soil immediately beneath the pile tip has been sheared to its critical state therefore  $\phi'$  goes to  $\phi'_{cv}$  and  $\alpha = (45 + \phi'_{cv}/2)$

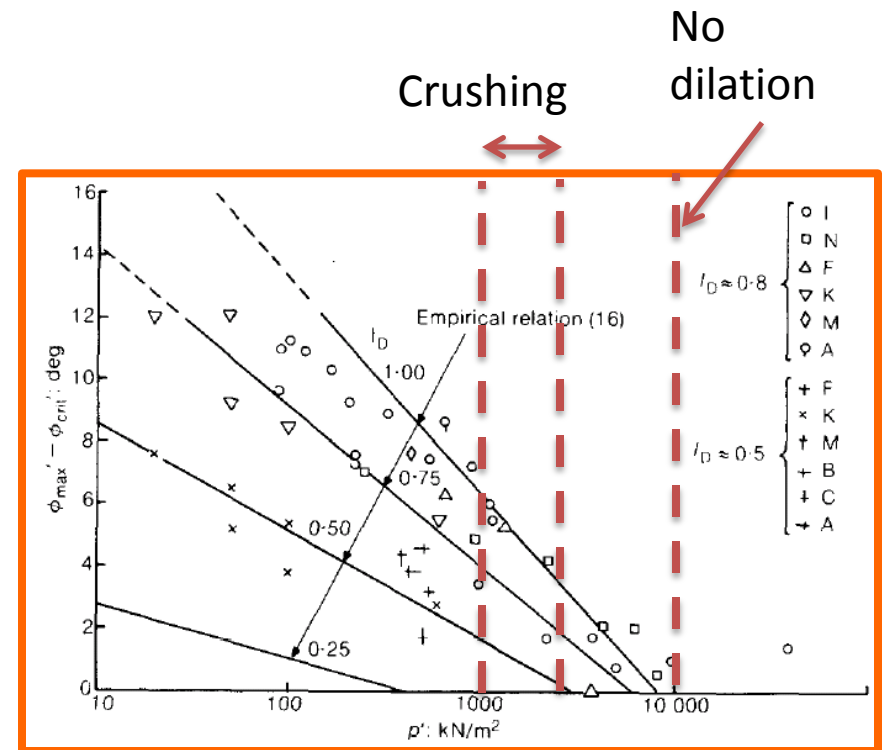


After Randolph



# DILATANCY

- Sands have been found to begin crushing at pressures ranging from 0.15 to 0.58 ksi (Bolton).
- The Triaxial data collected indicates zero dilation for the materials tested at approximately 1.45 ksi (10,000 kN/m<sup>2</sup>)

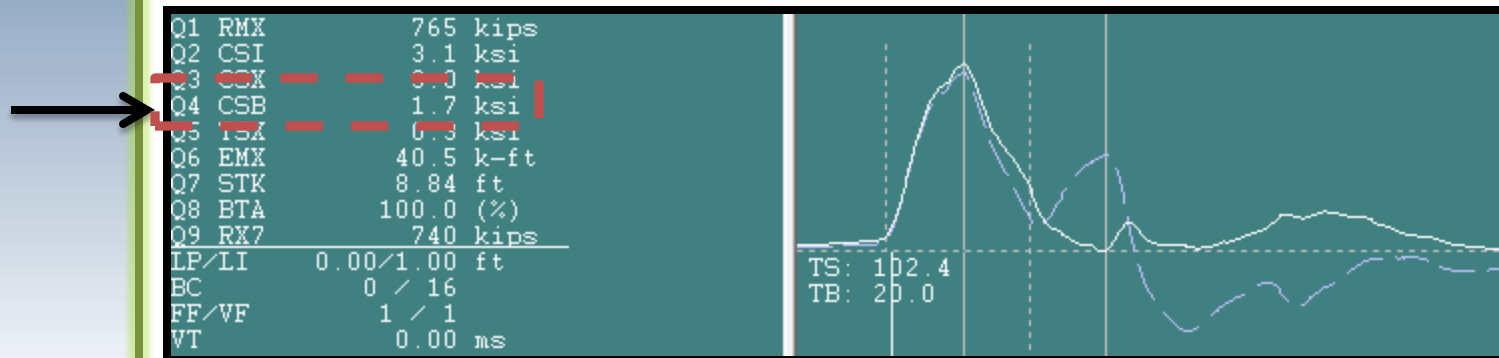
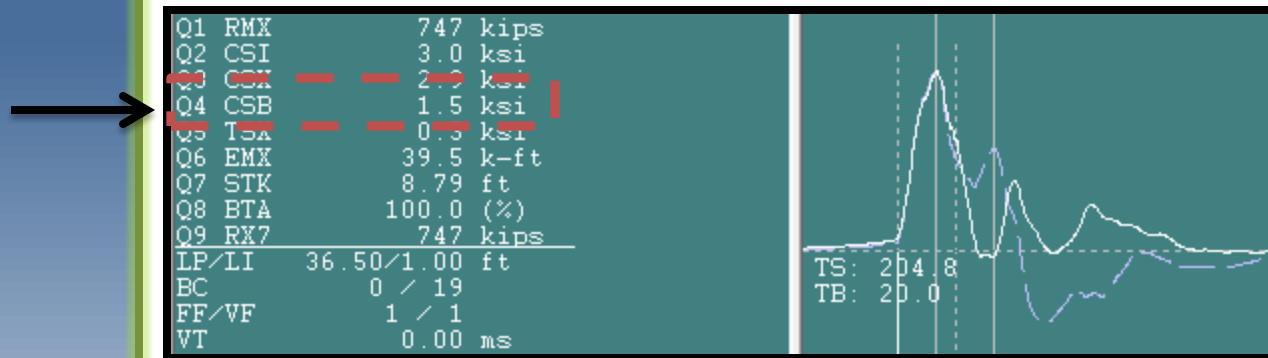


Triaxial data of Lee & Seed



# DILATANCY

- Tip stresses from two different piles in sands



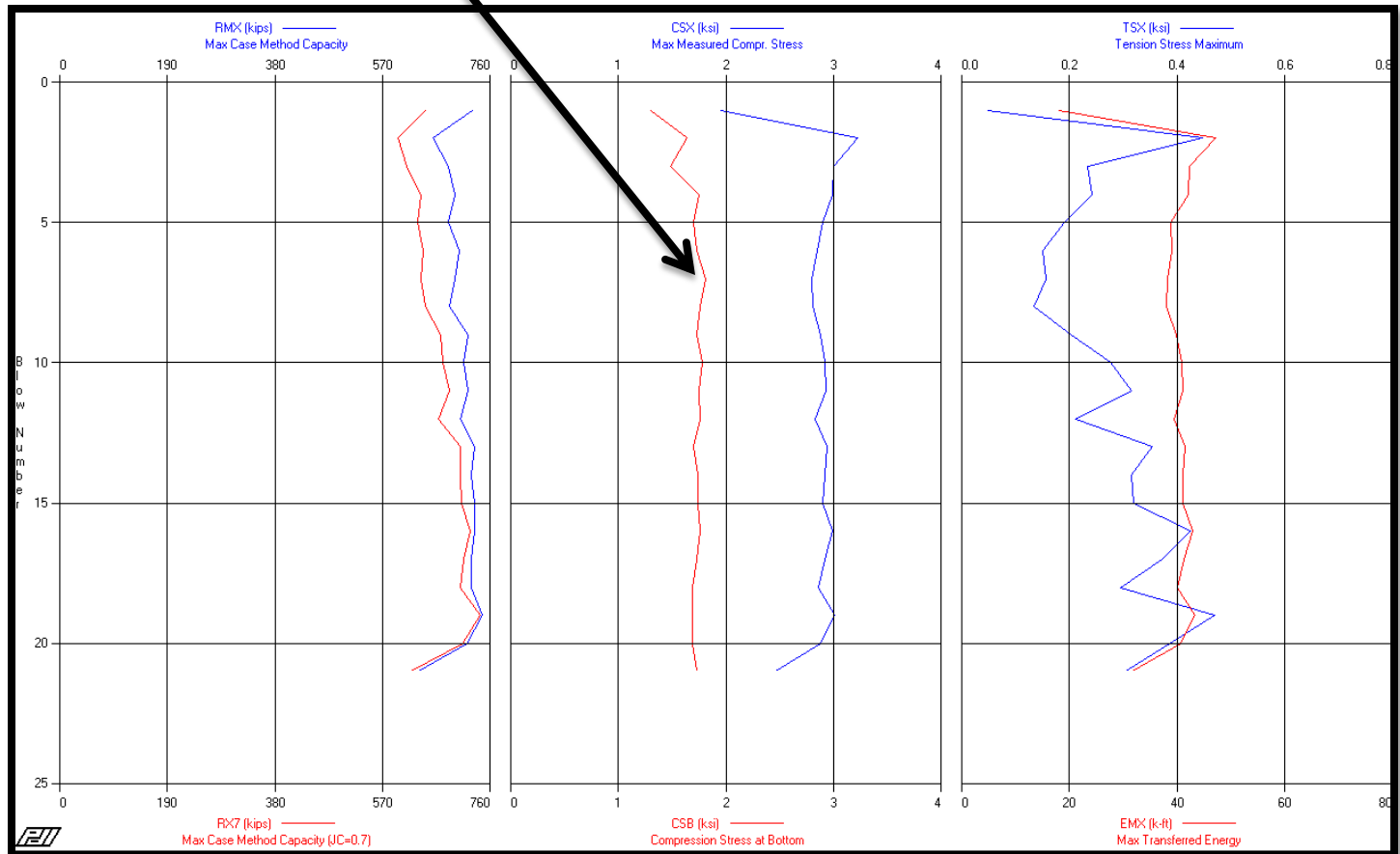
Tip stresses from PDA (CSB) > 1.45 ksi >> 0.58 ksi

Possible soil crushing and zero dilation at the soil-pile tip interface



# DILATANCY

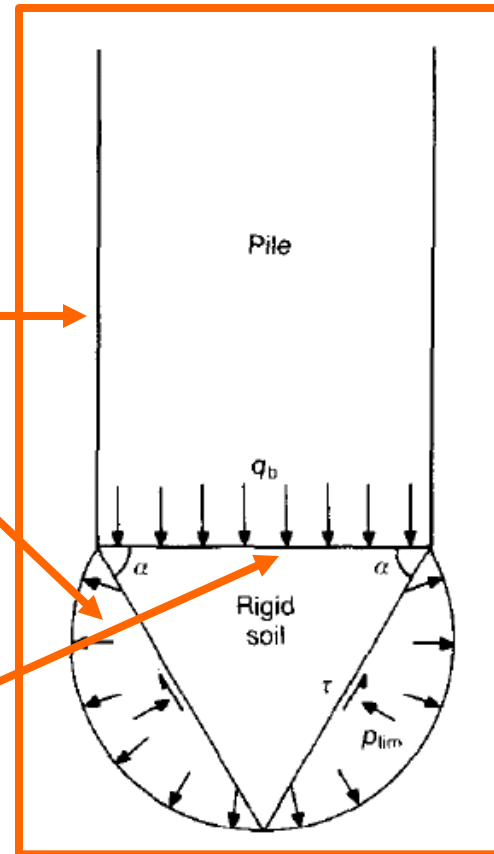
Tip stress (CSB)  $\approx 1.7$  ksi on average





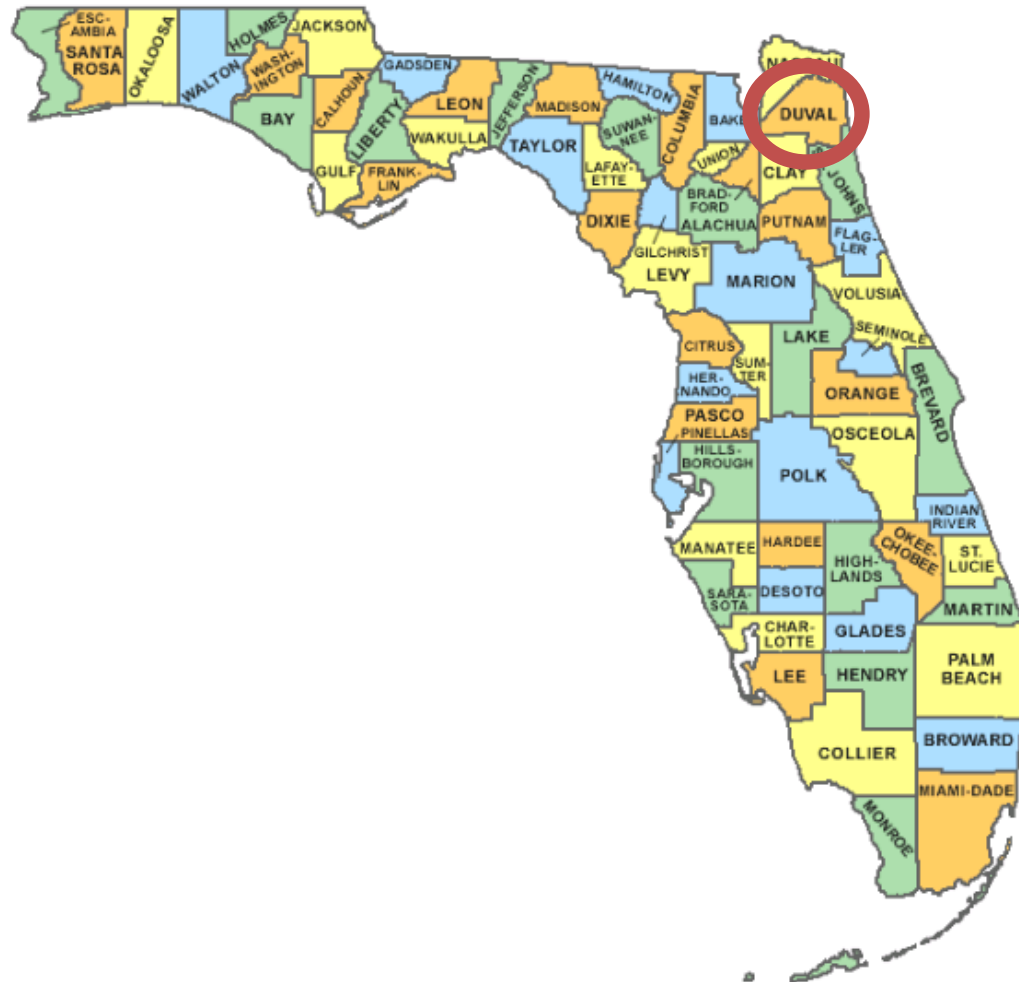
# DILATANCY

- A certain distance away from the pile tip and along the side,  $\psi$  reaches a maximum value, dilation occurs and negative pore pressures may develop
- Zone of Critical State where crushing occurs, friction =  $\phi'_{cv}$  and  $\psi = 0$



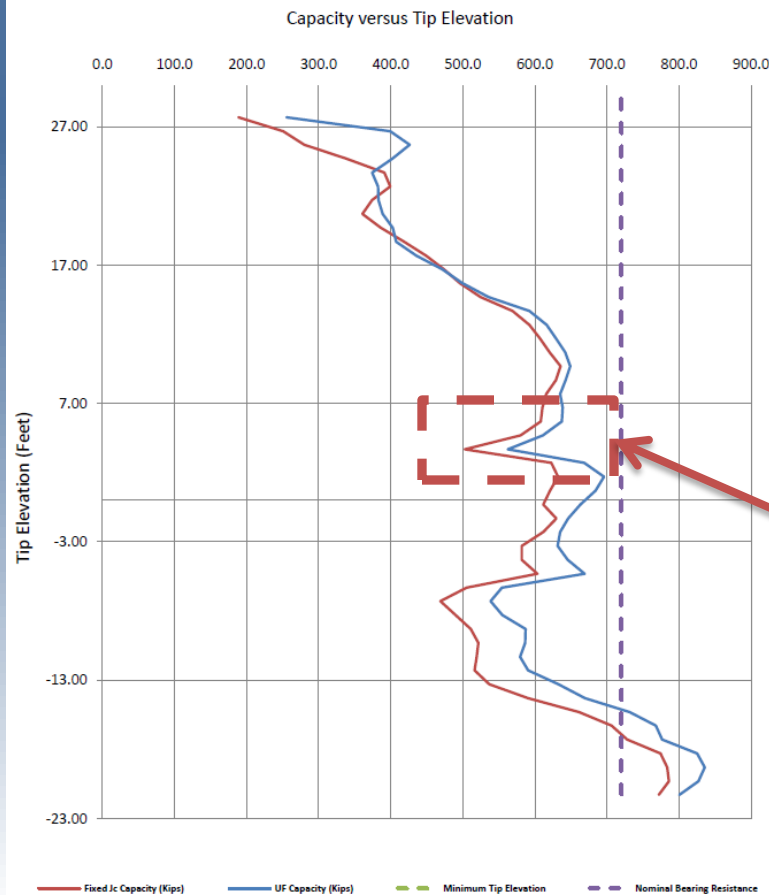


# DILATANCY

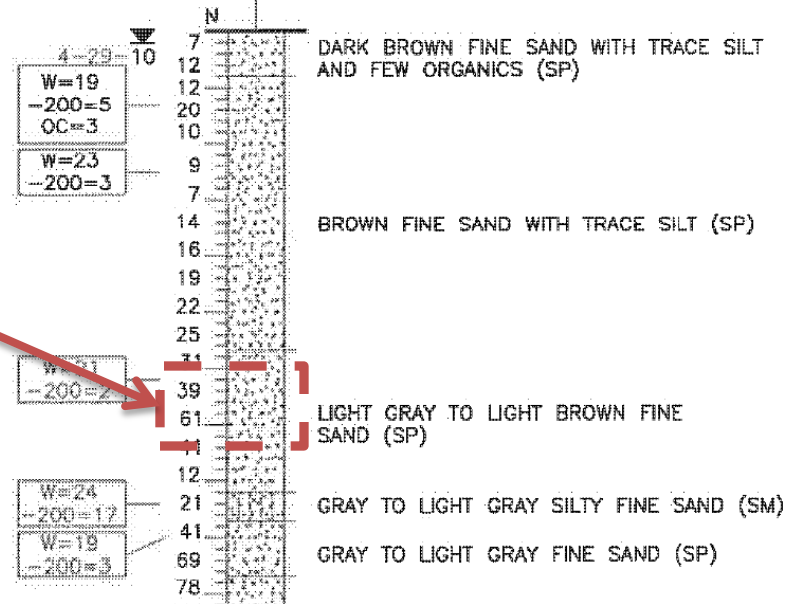




# DILATANCY



BORING No.: BB-2  
STATION: 2400+72  
OFFSET: 55' LT.  
ELEVATION: +35.2'



EDC data



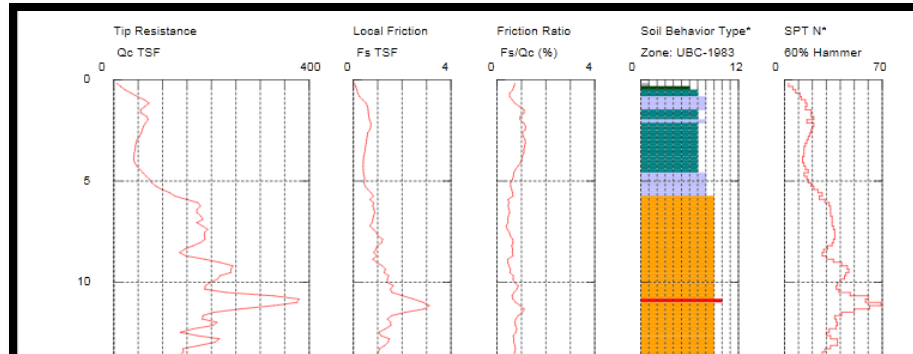
# PIEZOCONE

- CPT-U  
performed at  
bridges where  
soil relaxation  
was  
encountered





# PIEZOCONE

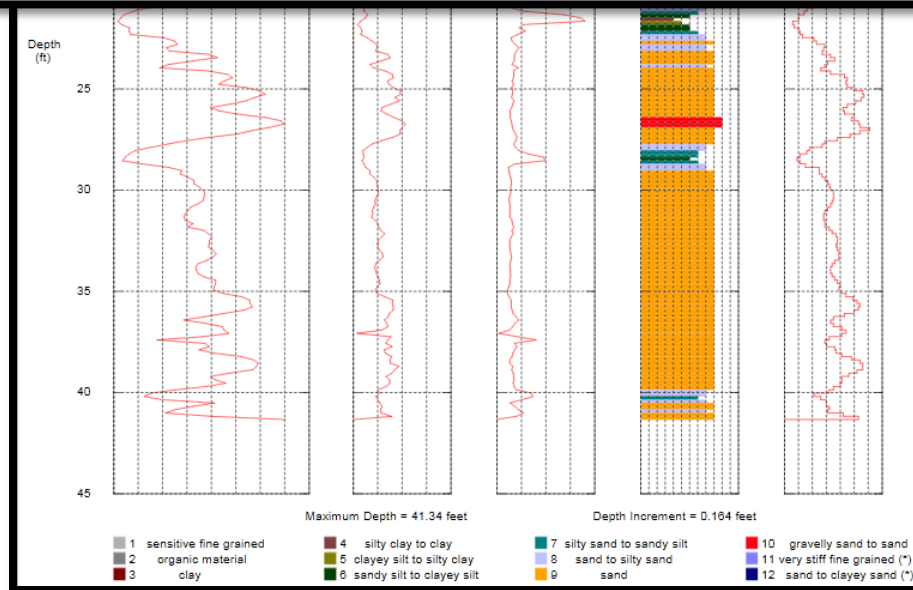


- 1 sensitive fine grained
- 2 organic material
- 3 clay

- 4 silty clay to clay
- 5 clayey silt to silty clay
- 6 sandy silt to clayey silt

- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand

- 10 gravelly sand to sand
- 11 very stiff fine grained (\*)
- 12 sand to clayey sand (\*)



- 1 sensitive fine grained
- 2 organic material
- 3 clay

- 4 silty clay to clay
- 5 clayey silt to silty clay
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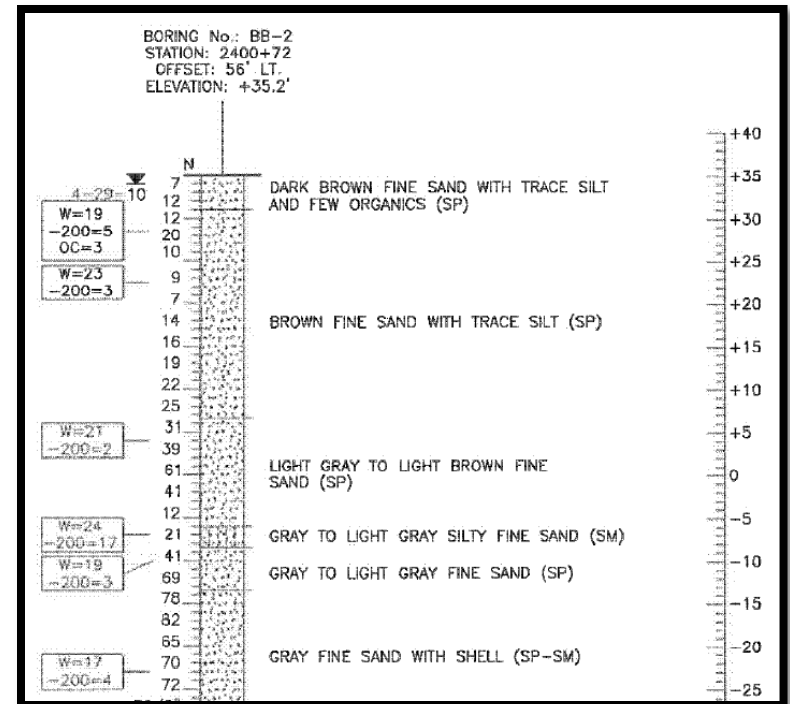
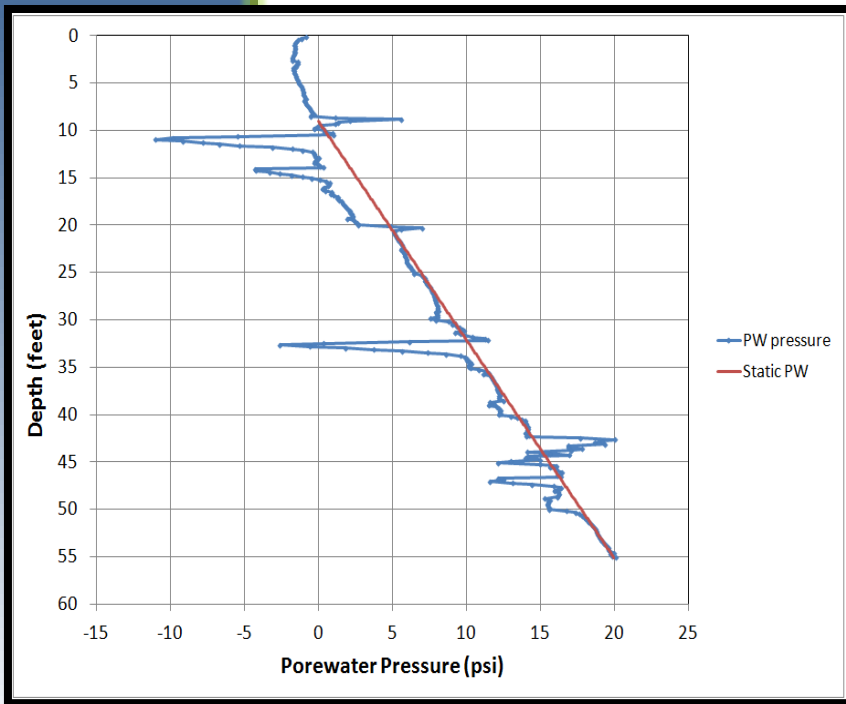
- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand

- 10 gravelly sand to sand
- 11 very stiff fine grained (\*)
- 12 sand to clayey sand (\*)



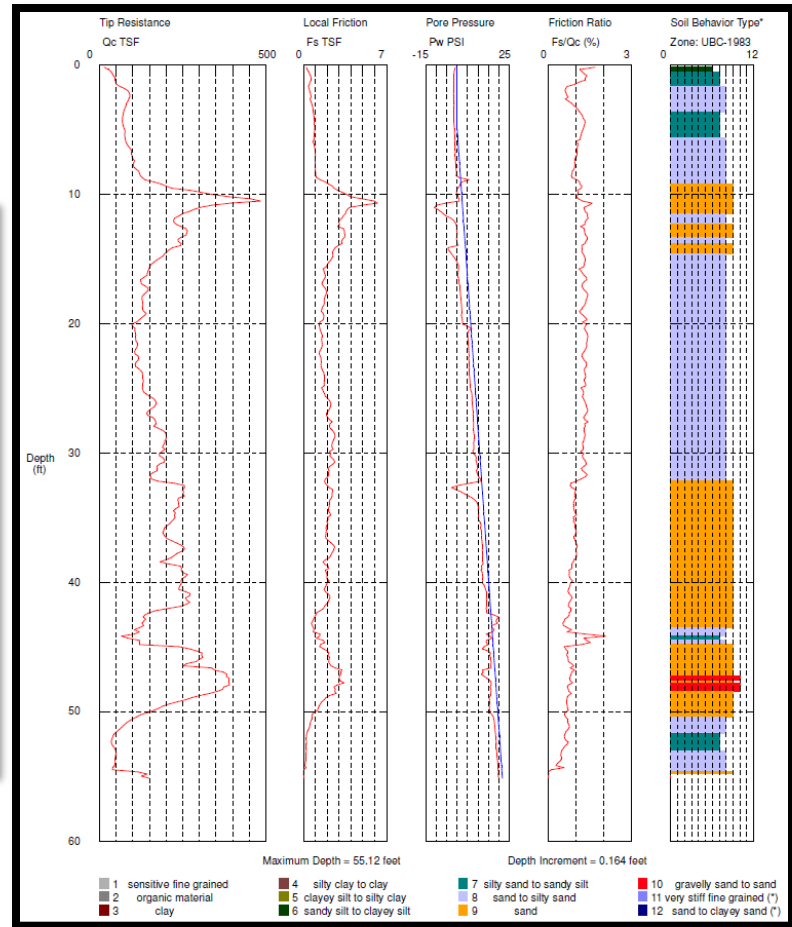
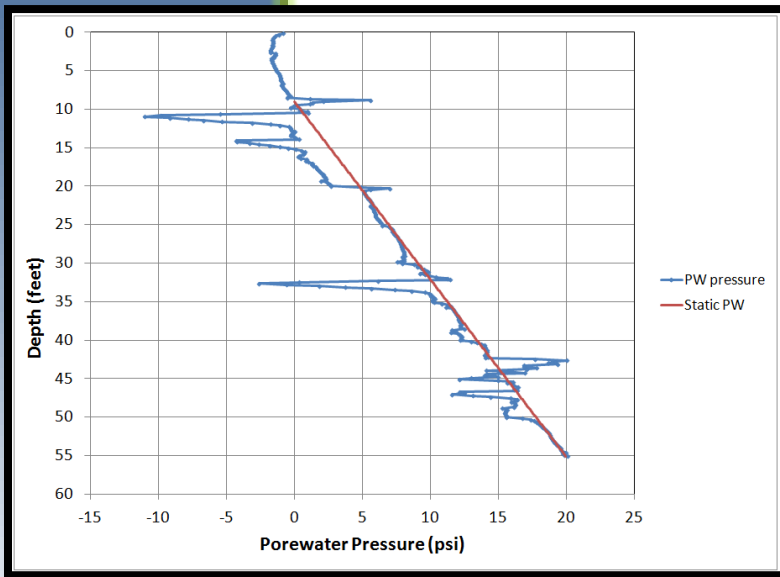
# PIEZOCONE

## Safety Hammer



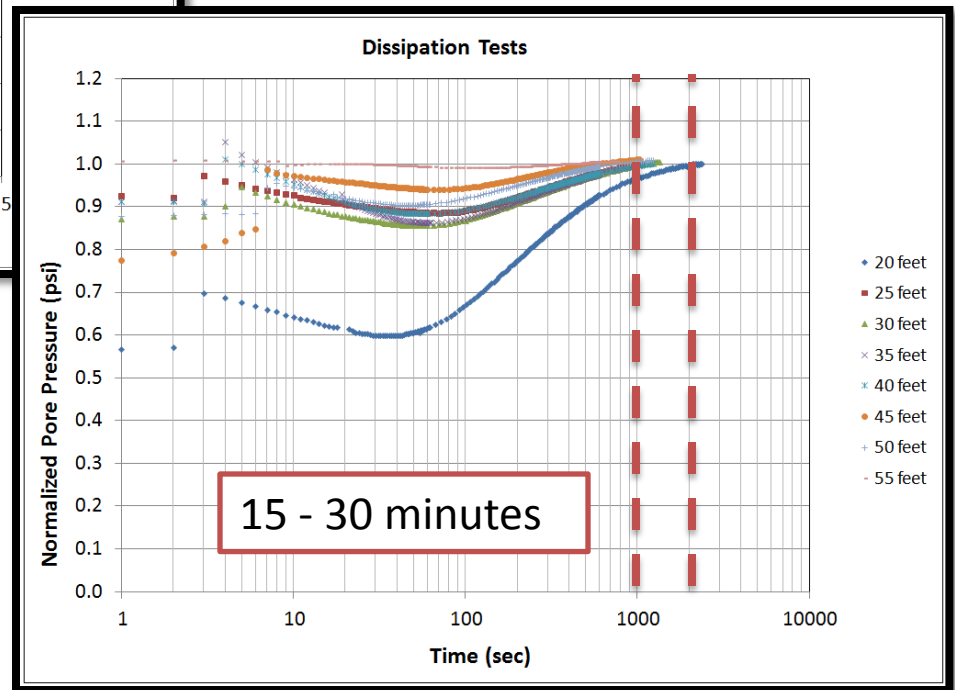
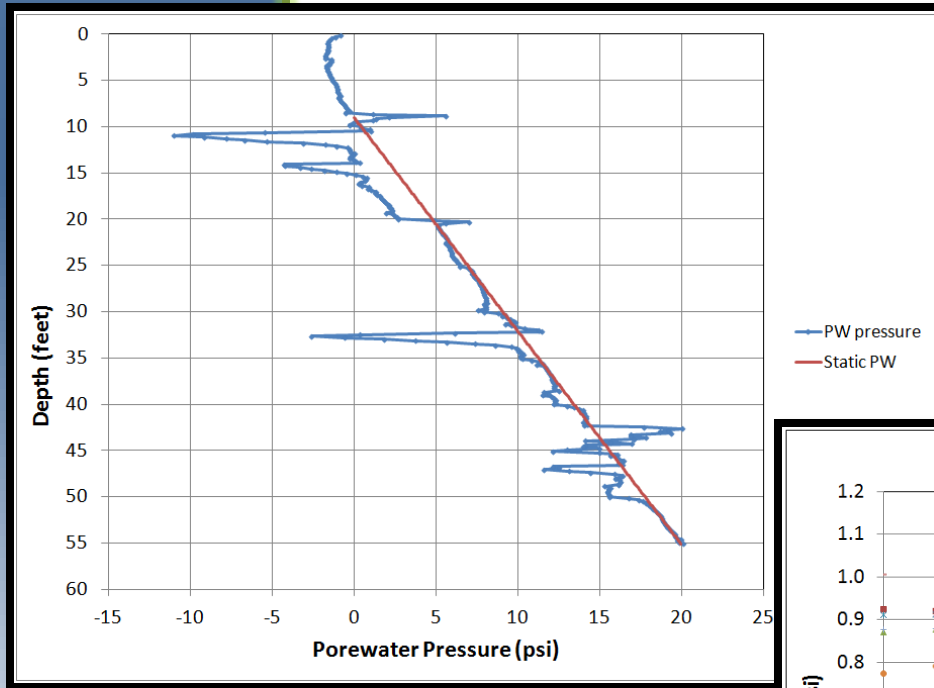


# PIEZOCONE



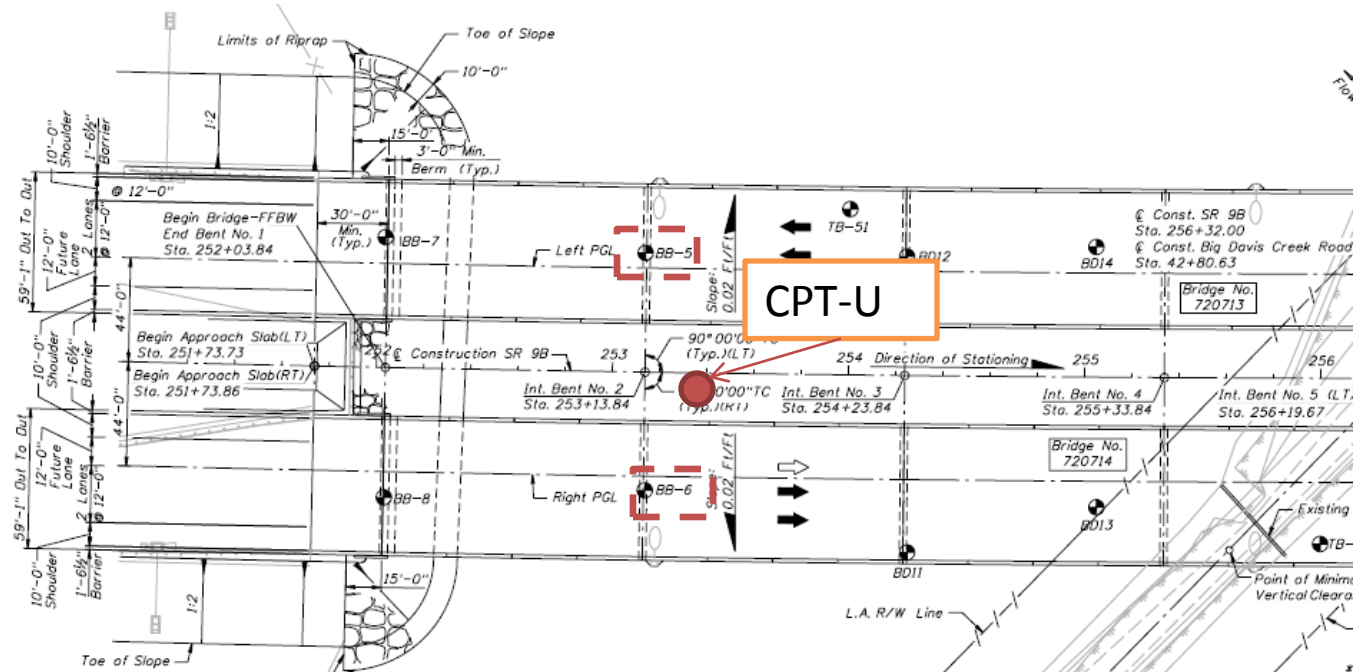


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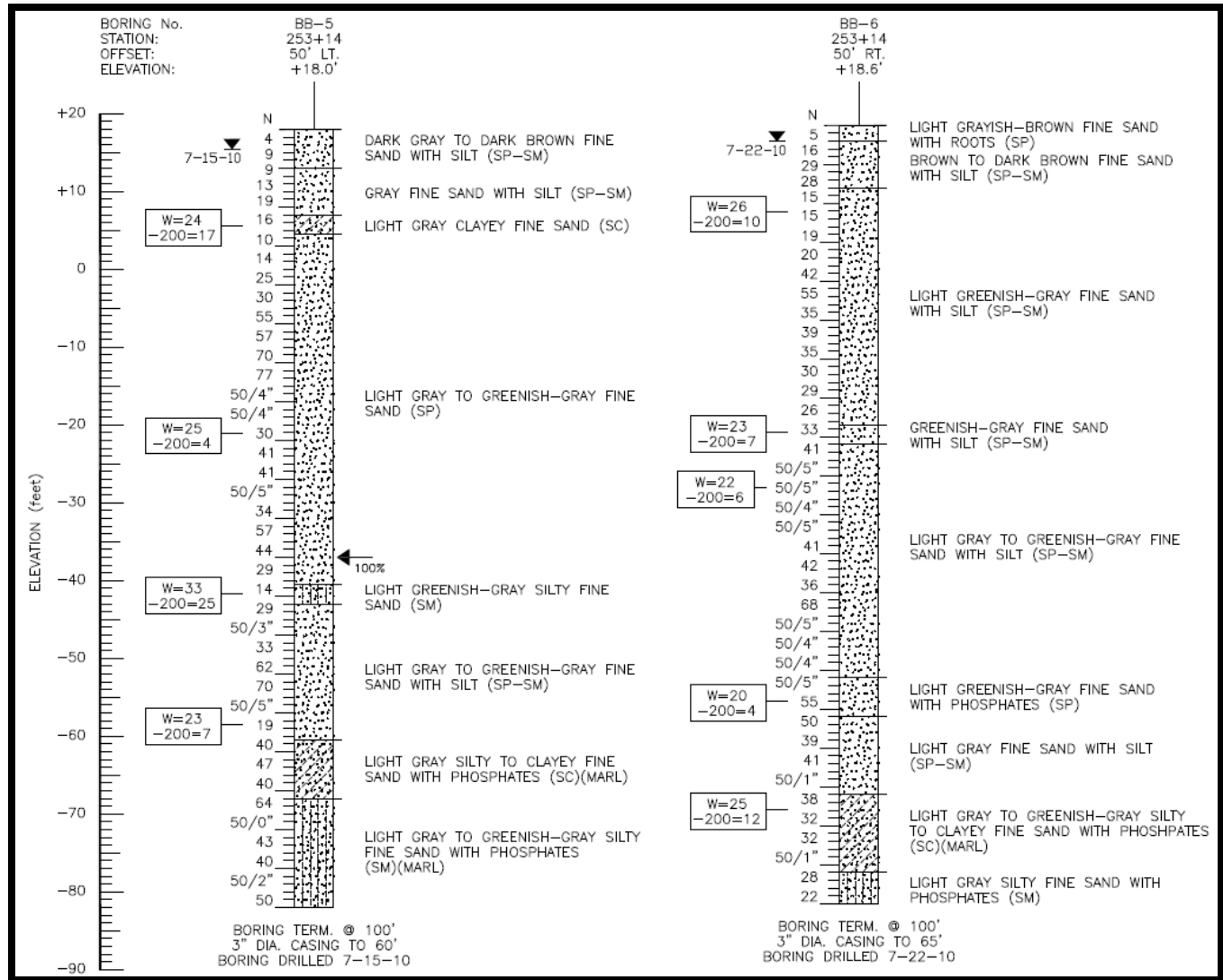


# PIEZOCONE



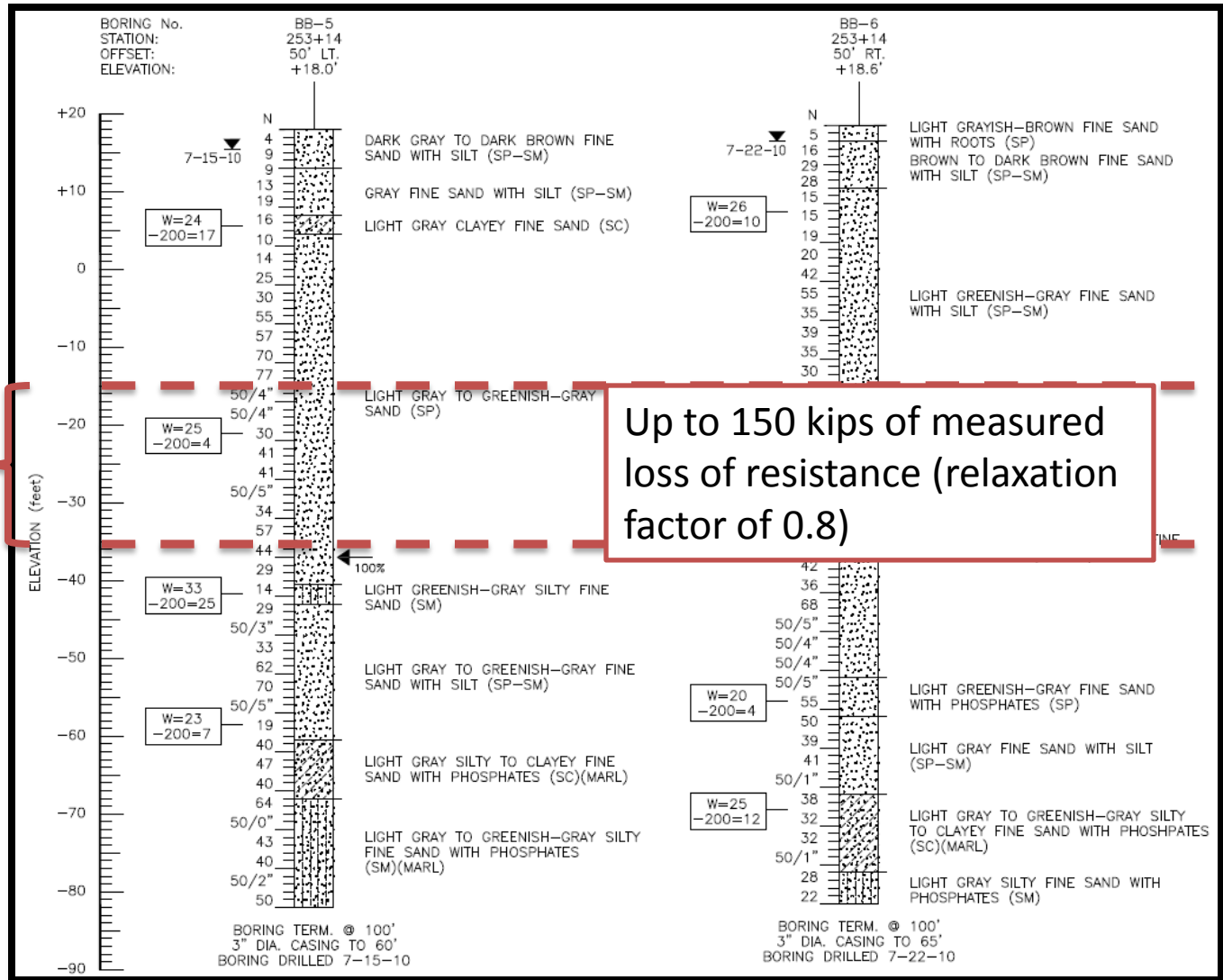


# PIEZOCONE





# PIEZOCONE

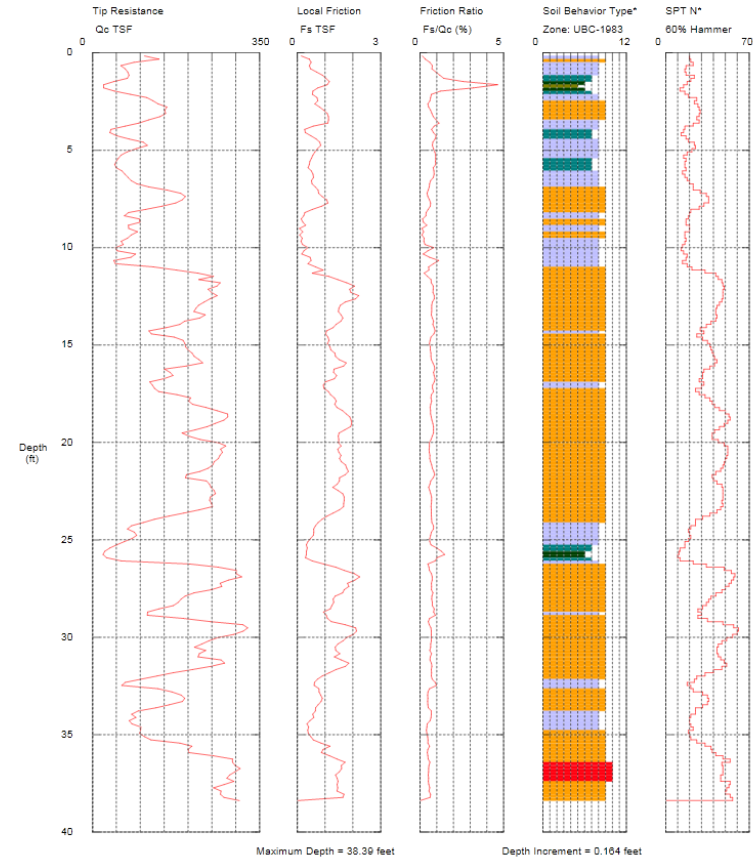
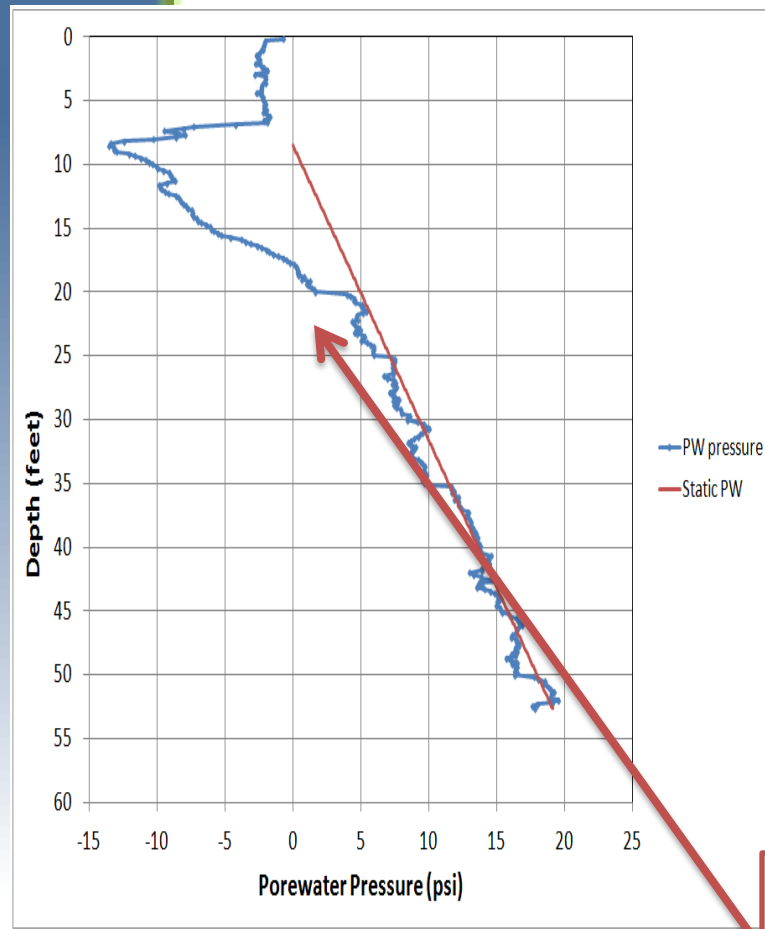


Zone of  
measured  
relaxation

Up to 150 kips of measured loss of resistance (relaxation factor of 0.8)



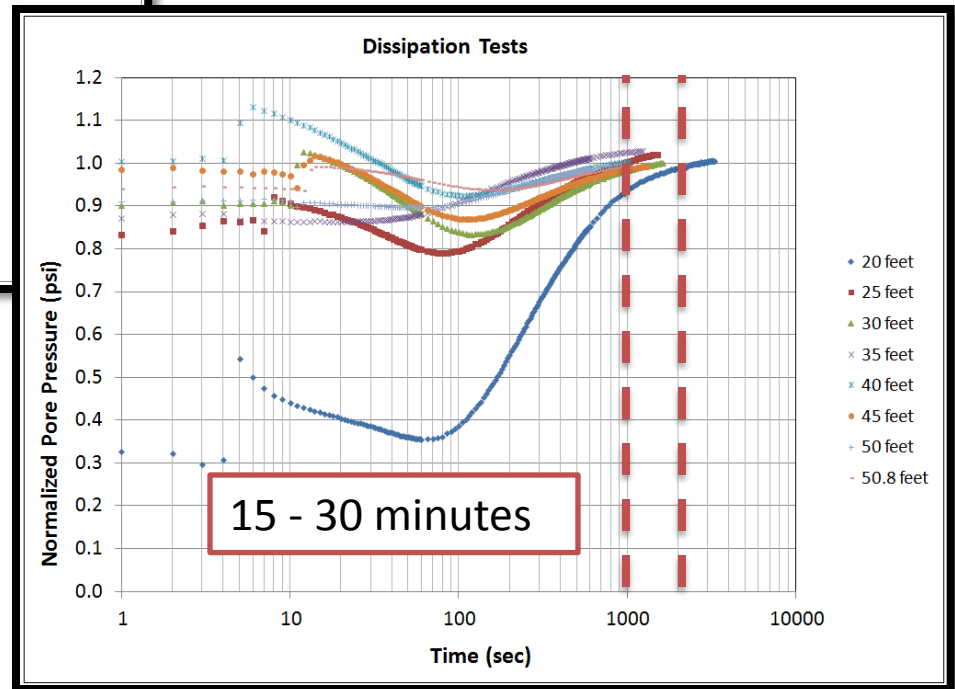
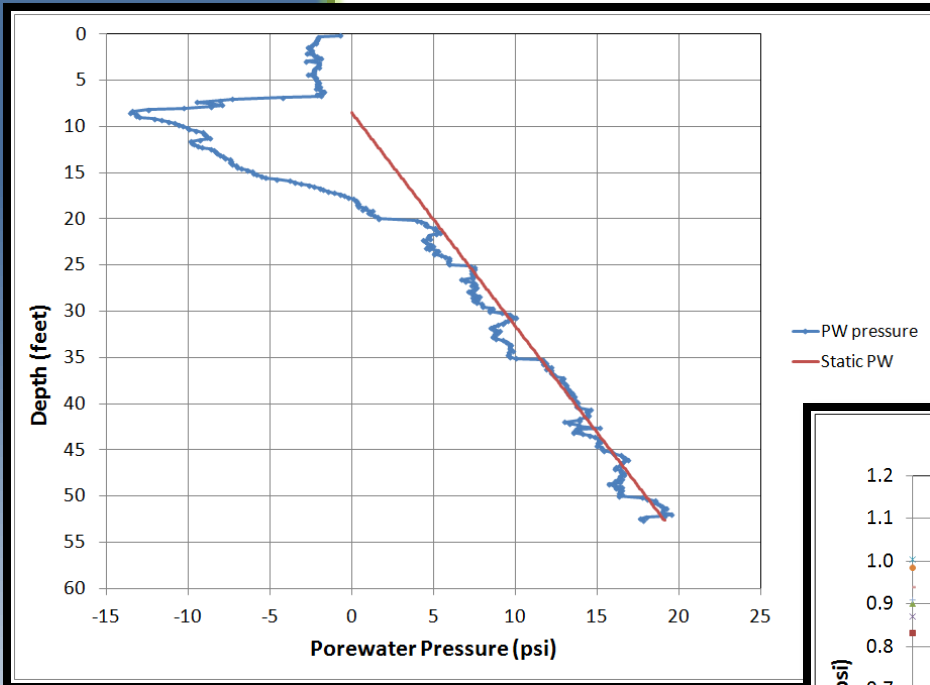
# PIEZOCONE



At approximately 20mm/second penetration rate (0.066 ft/second).  
Pile velocity  $\approx$  4 to 8 ft/sec.

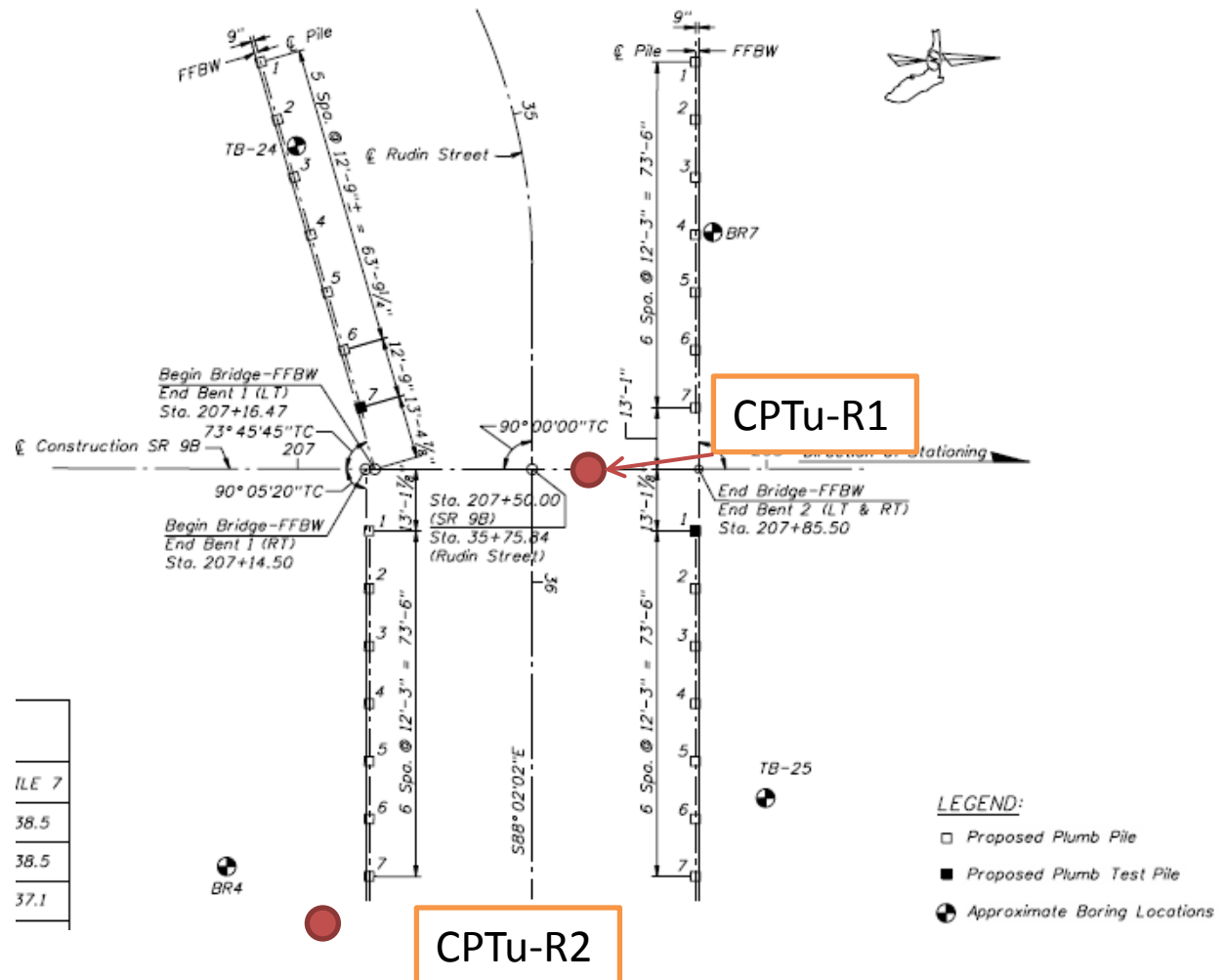


# PIEZOCONE





# PIEZOCONE

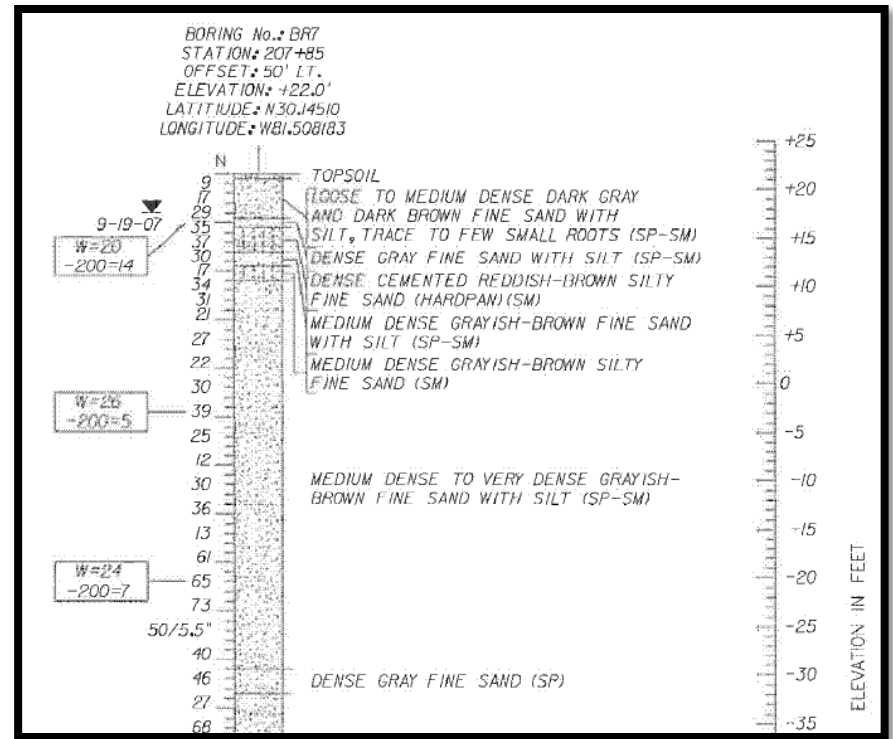
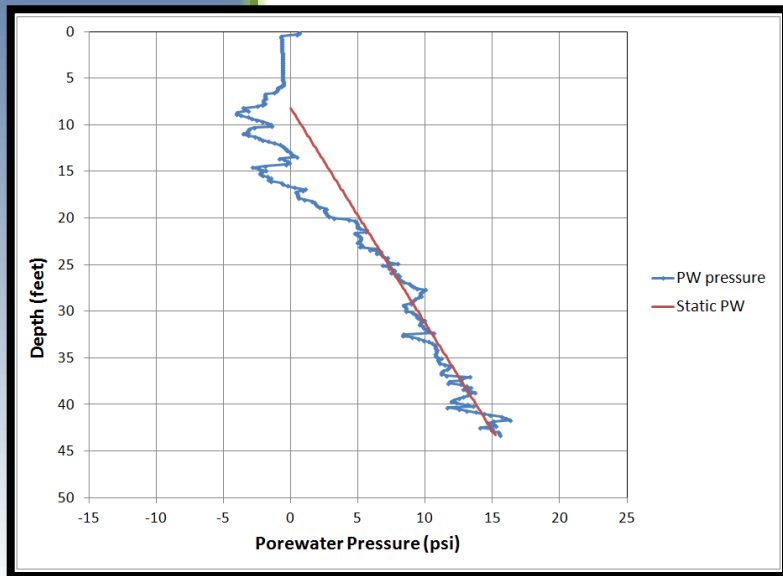


ILE 7
38.5
38.5
37.1



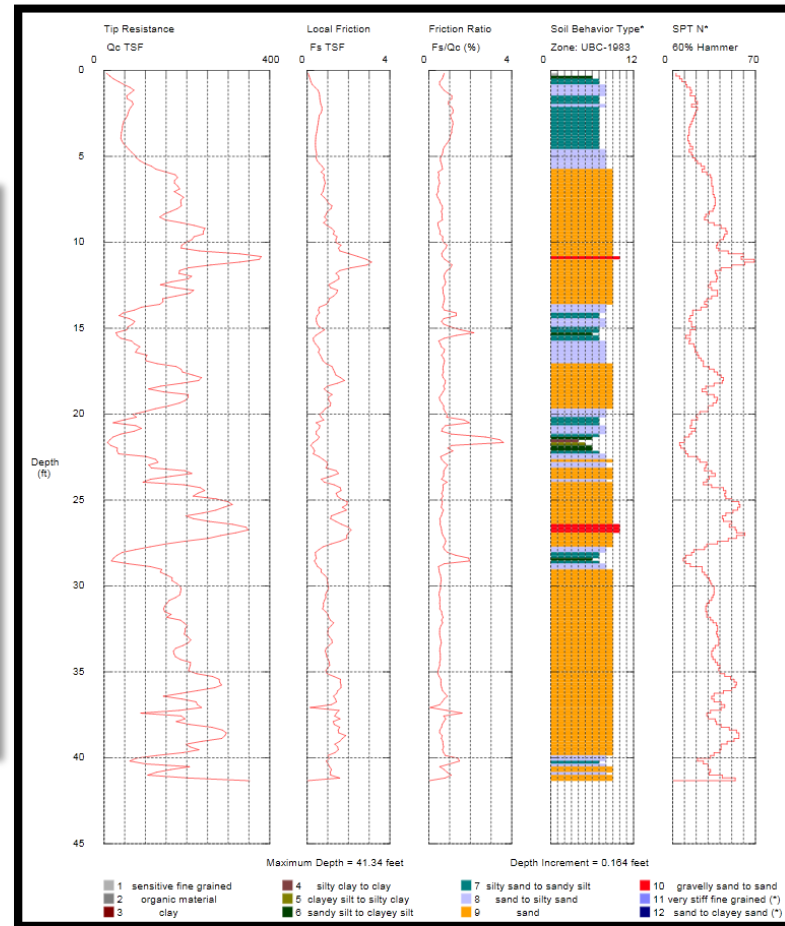
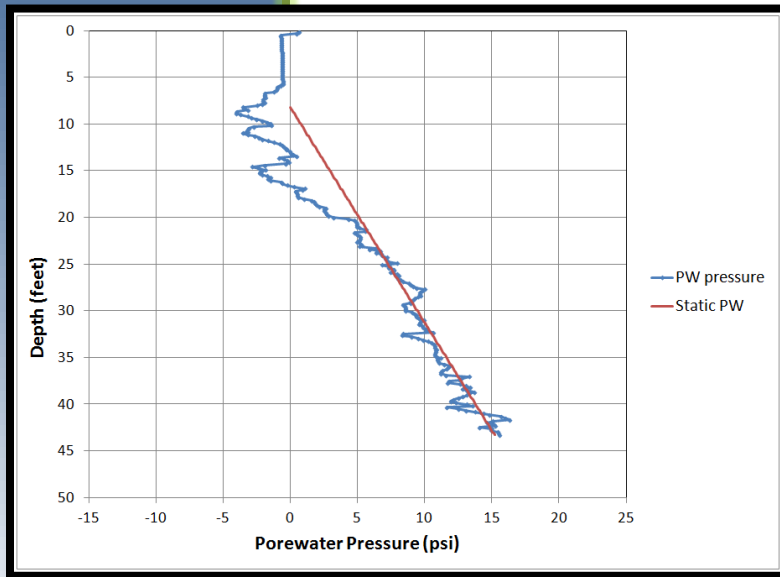
# PIEZOCONE

## Safety Hammer



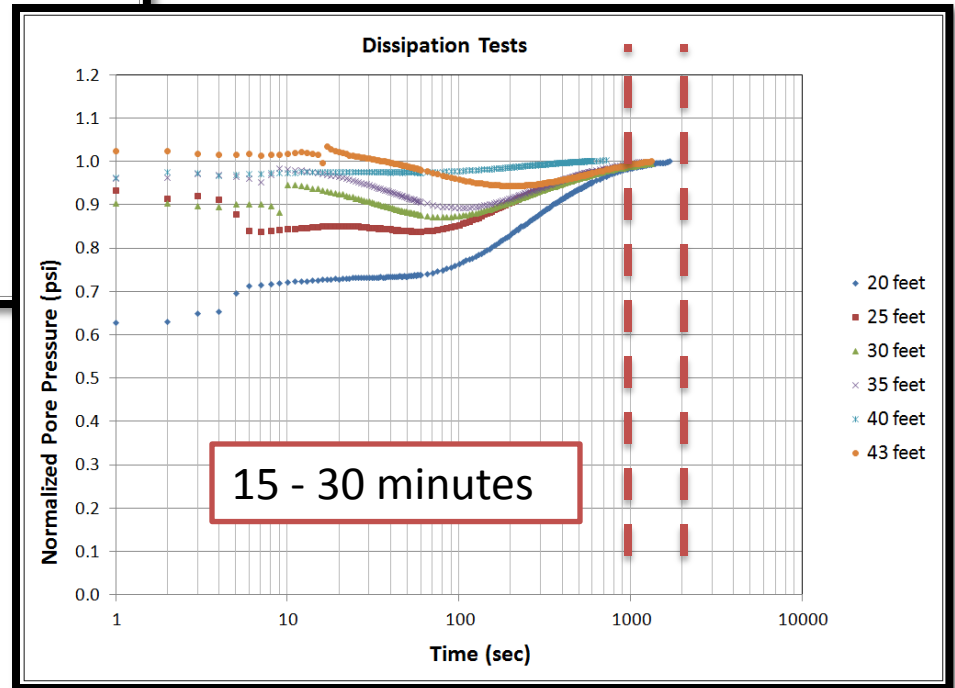
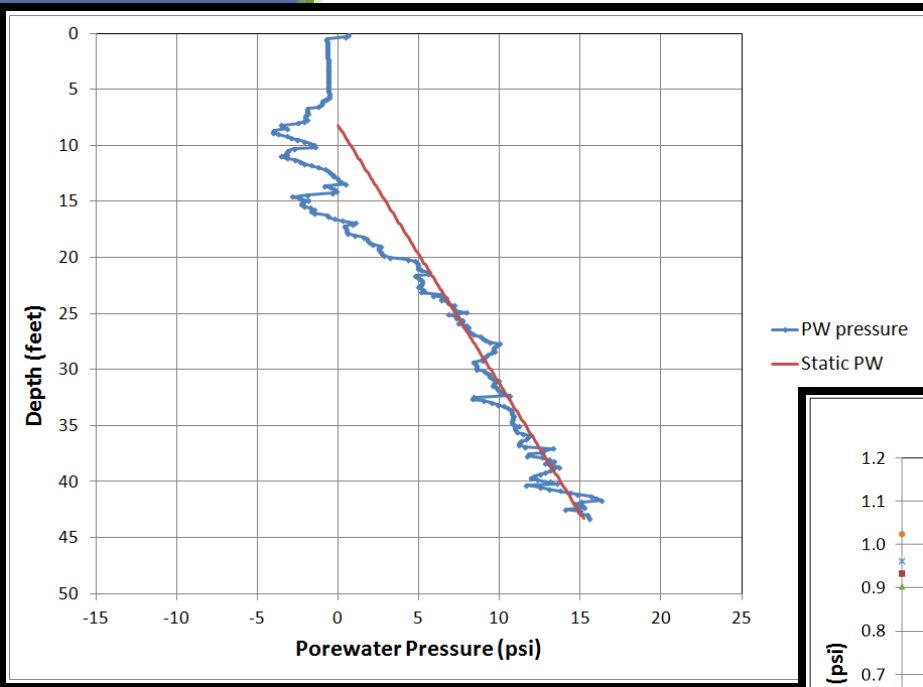


# PIEZOCONE





# PIEZOCONE





# DILATANCY



SMO's CPTU

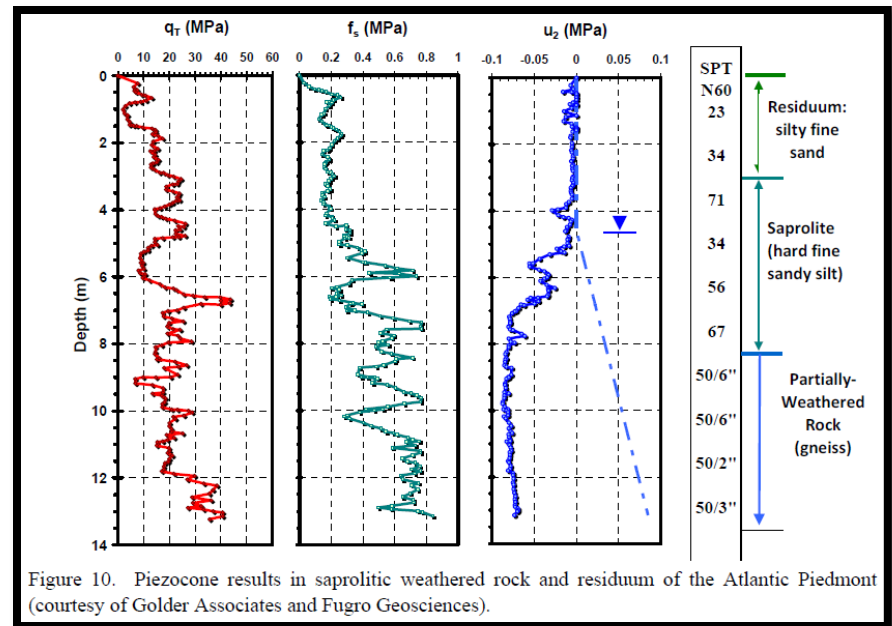
Shoulder transducer





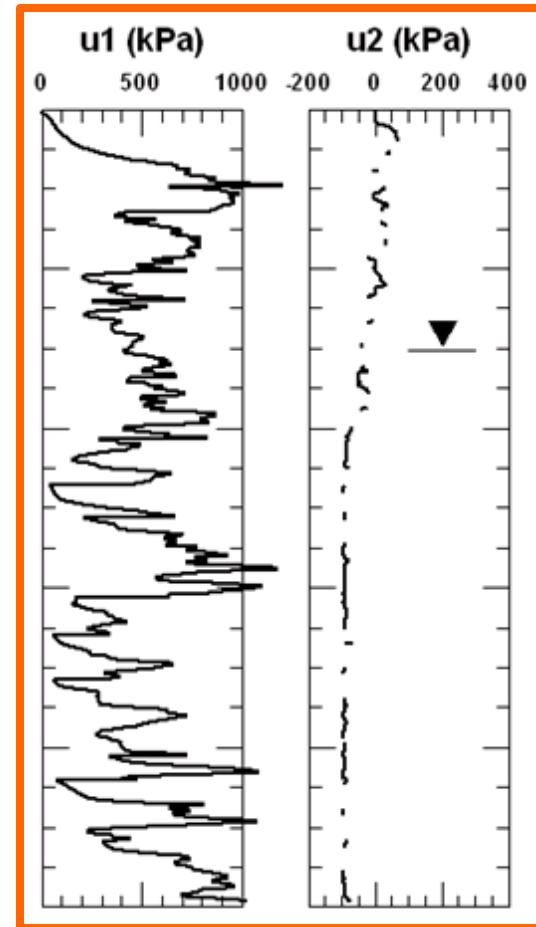
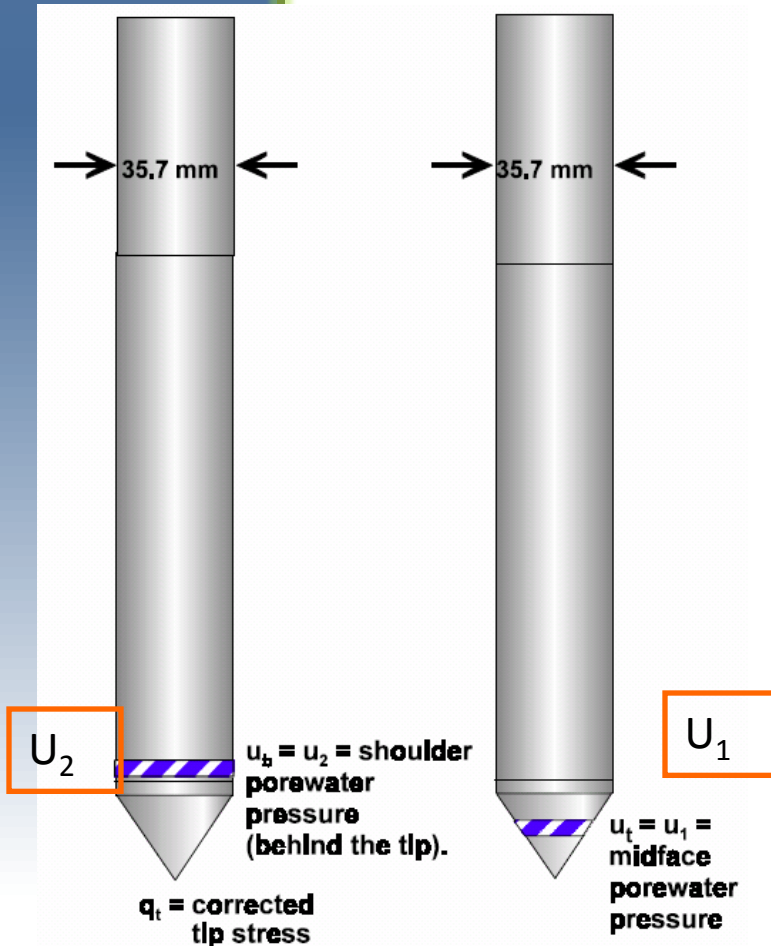
# DILATANCY

- CPTs can be used to penetrate hard materials, including partially-weathered rocks (Mayne 2010)





# DILATANCY



U2 position is required



# DILATANCY

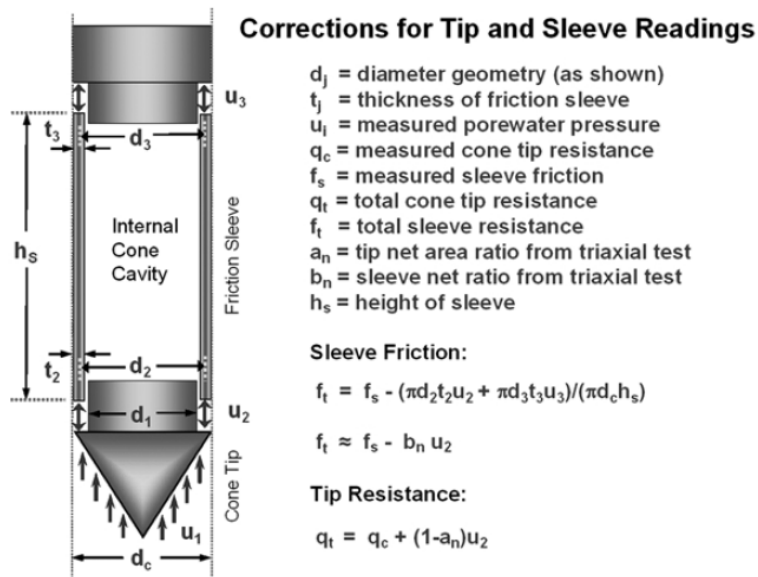


FIGURE 12 Determination of total cone tip resistance and total sleeve friction (Jamiolkowski et al. 1985).



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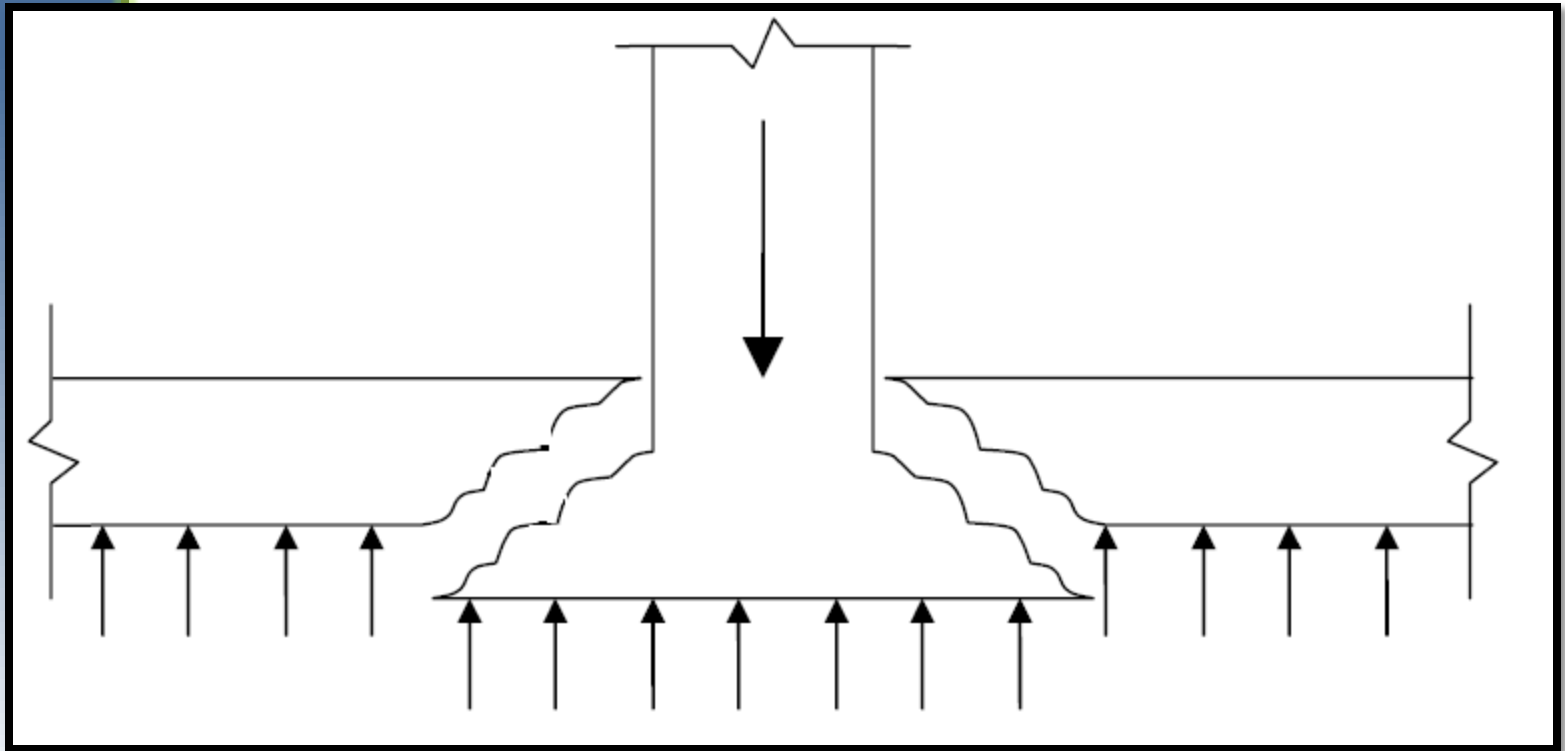


# DILATANCY - SUMMARY

- Measured loss of pile resistance due to relaxation in Florida sands
  - Relaxation factors up to 0.8
- Use the piezocone during the preliminary investigation to identify potential areas of relaxation
- Construction requirements
  - Set checks
  - Minimum pile embedment
  - Verification testing

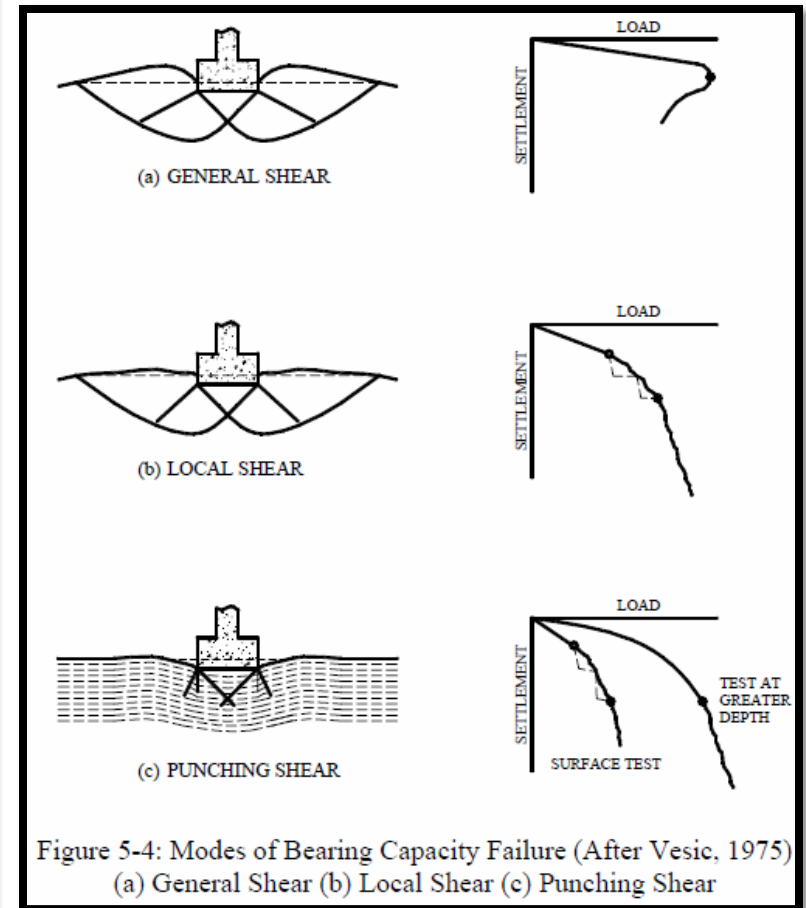
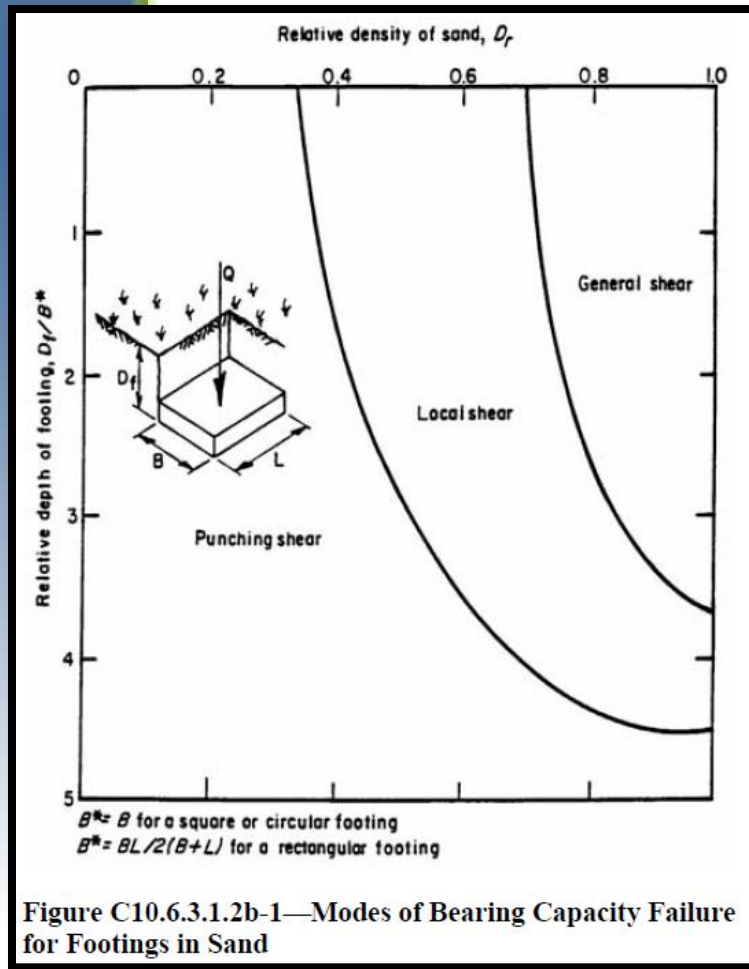


# PUNCHING SHEAR





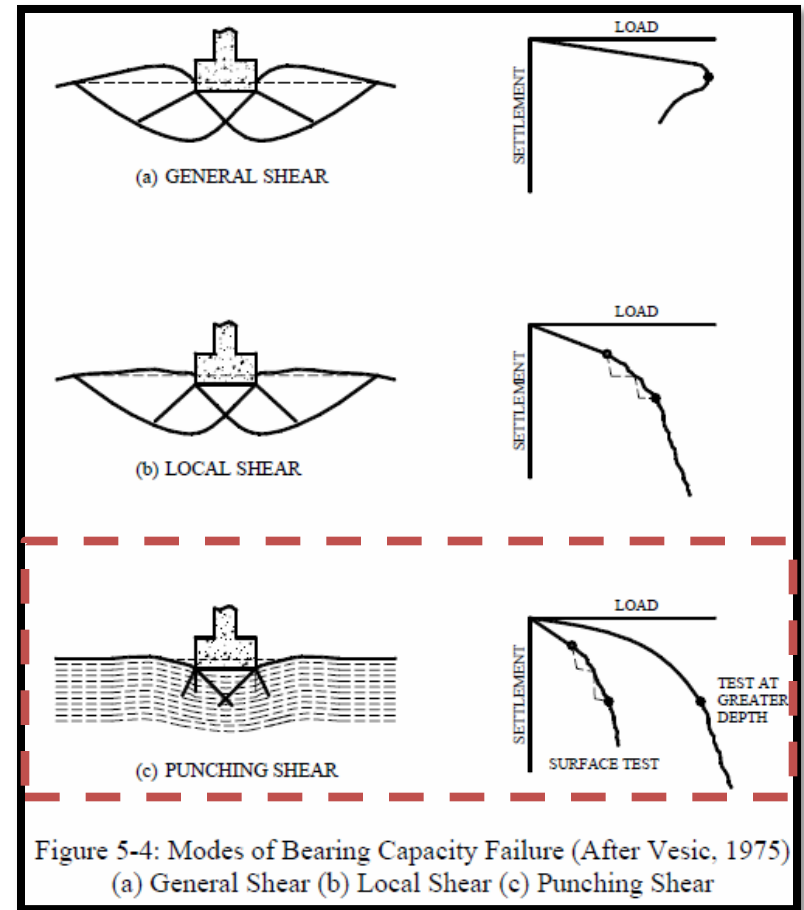
# PUNCHING SHEAR





# PUNCHING SHEAR

- Characterized by vertical shear around the perimeter of the footing, accompanied by a vertical movement and compression of the soil below the footing.





# PUNCHING SHEAR

$$c^* = 0.67c \quad (10.6.3.1.2b-1)$$

$$\phi^* = \tan^{-1}(0.67 \tan \phi_f) \quad (10.6.3.1.2b-2)$$

where:

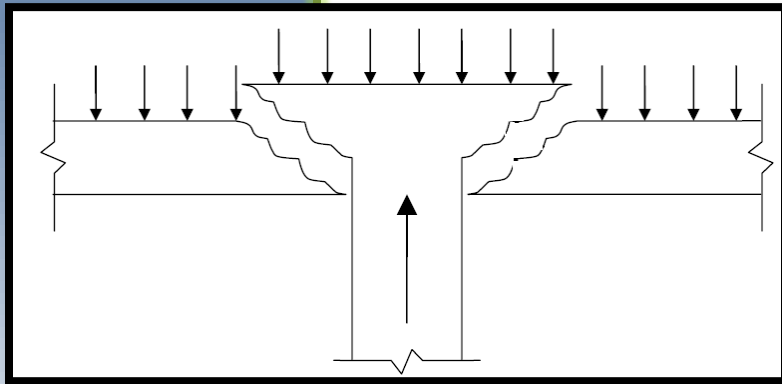
$c^*$  = reduced effective stress soil cohesion for punching shear (ksf)

$\phi^*$  = reduced effective stress soil friction angle for punching shear (degrees)

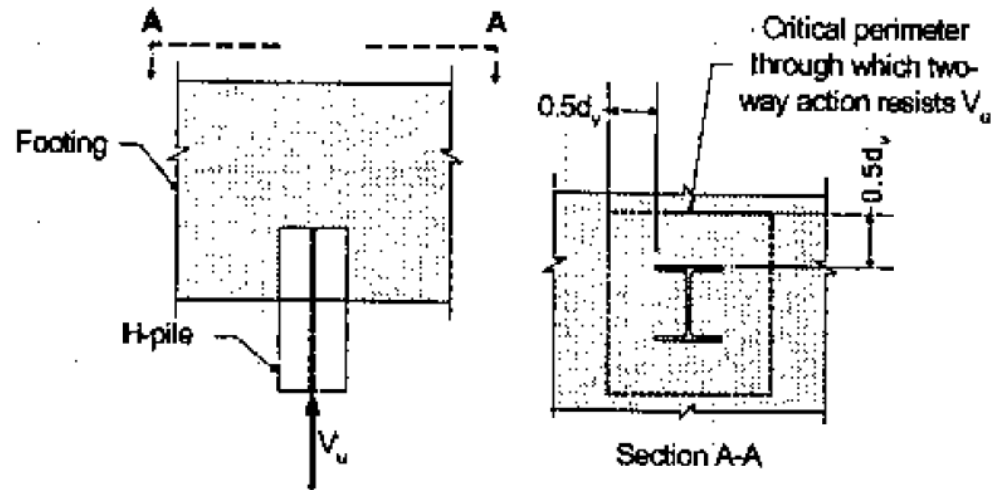
Reduced shear strength in Nominal Bearing Resistance calculation for Shallow Foundations (AASHTO)



# PUNCHING SHEAR



## Two-Way Action – Punching Shear



Pile Cap



# PUNCHING SHEAR

Two-way action without transverse reinforcement

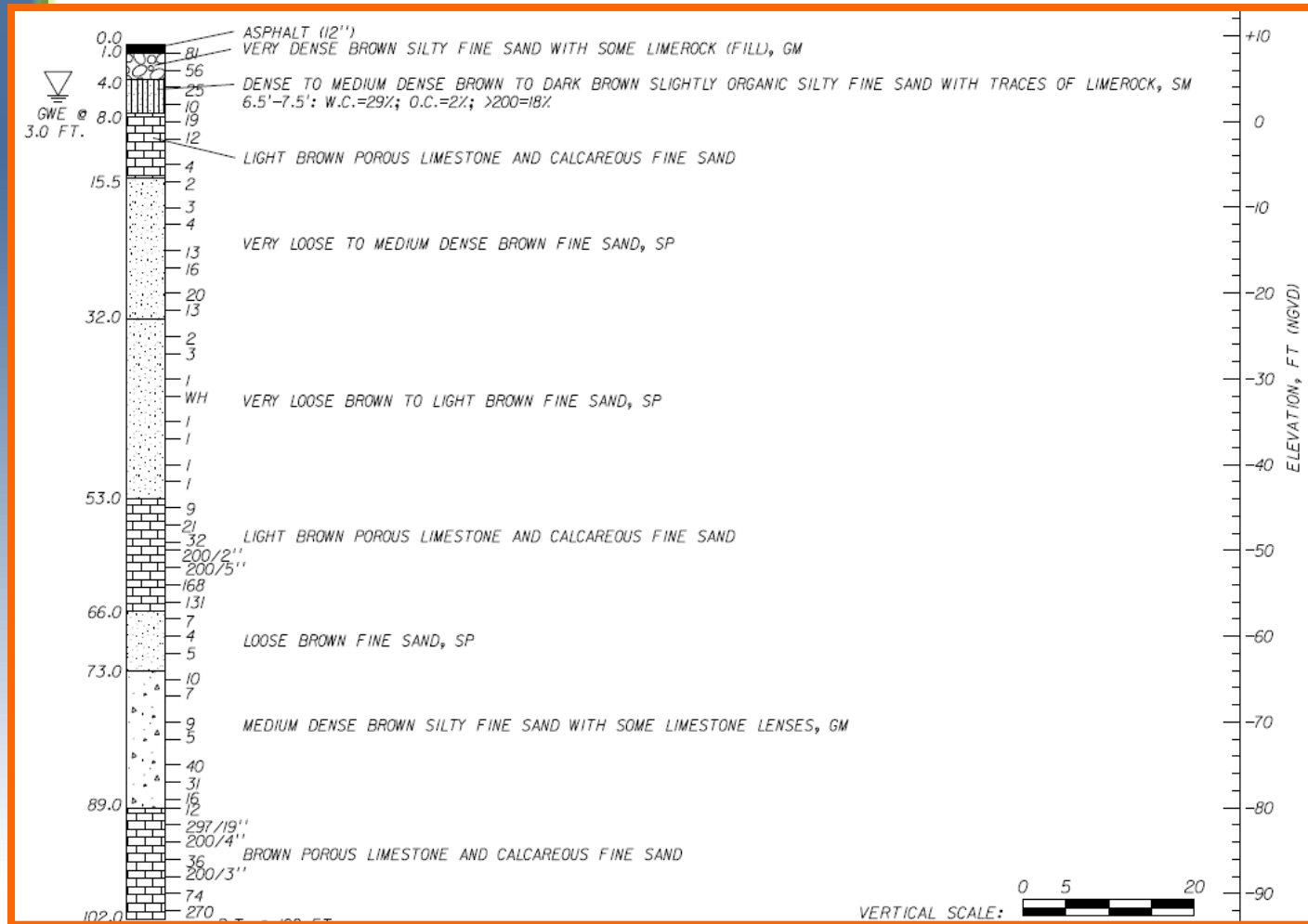
$$V_n = \left( 0.063 + \frac{0.126}{\beta_c} \right) \sqrt{f'_c} b_o d_v \leq 0.126 \sqrt{f'_c} b_o d_v$$

Two-way action with transverse reinforcement

$$V_n = V_c + V_s \leq 0.192 \sqrt{f'_c} b_o d_v$$



# PUNCHING SHEAR



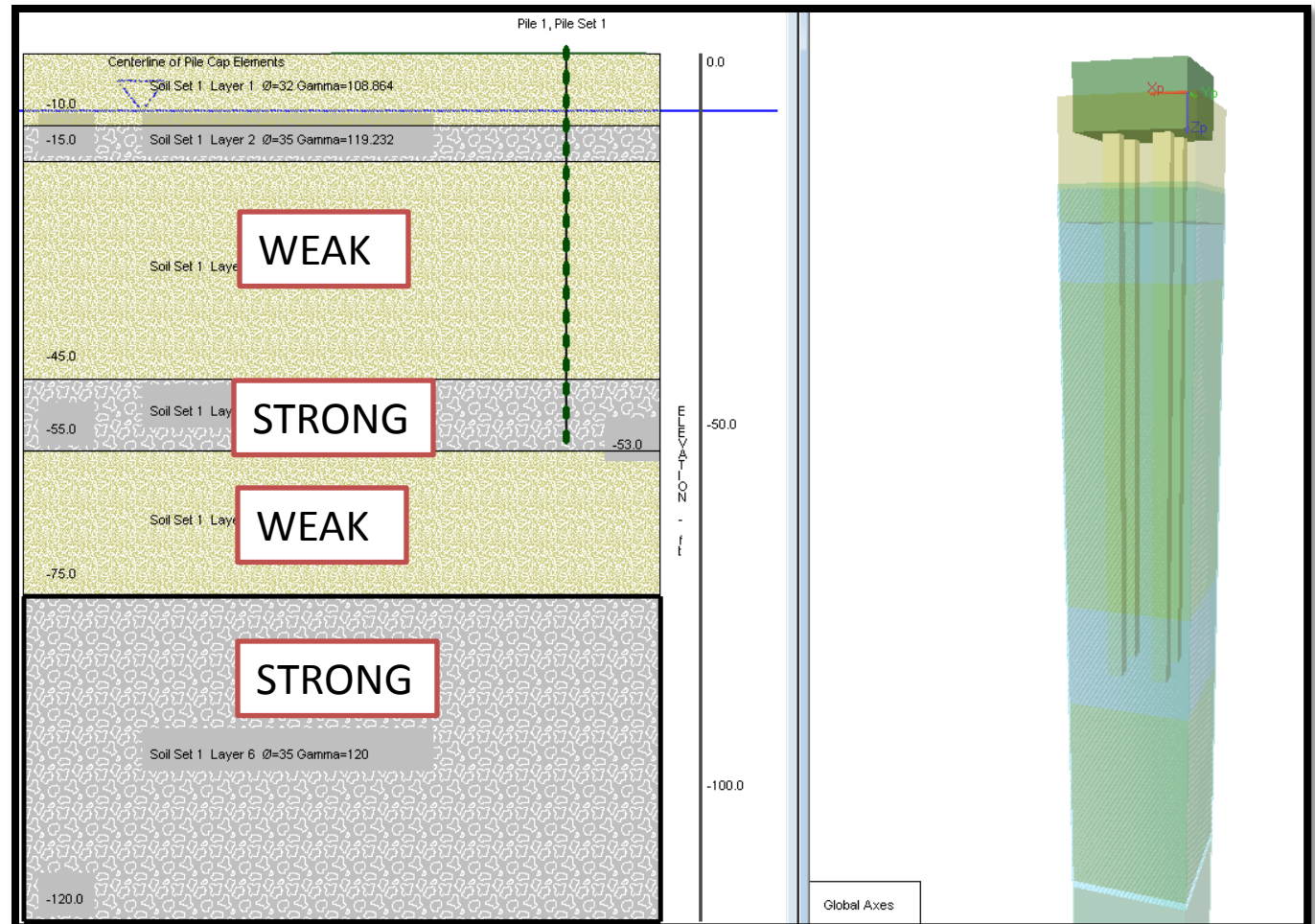


# PUNCHING SHEAR



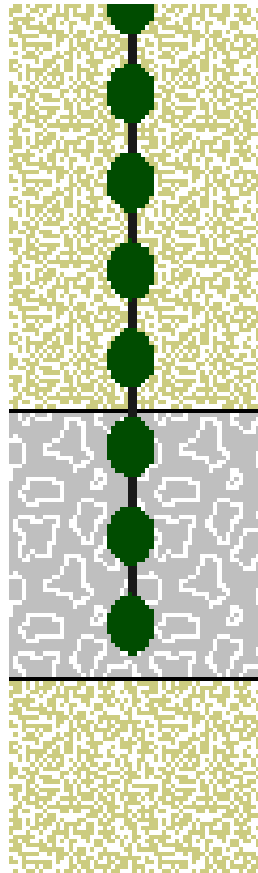


# PUNCHING SHEAR





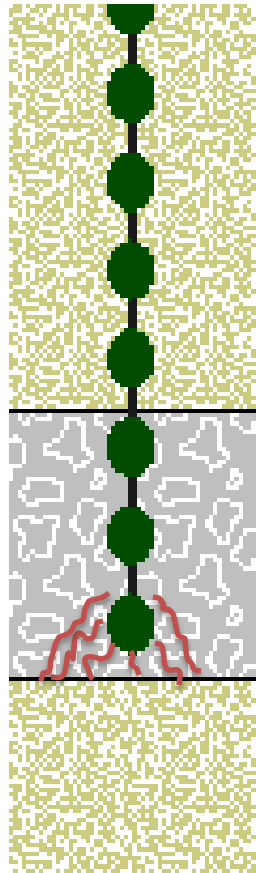
# PUNCHING SHEAR



Enough to support the group??



# PUNCHING SHEAR



Cracking of the rock mass during pile driving may result in an increased probability of punching



# PUNCHING SHEAR

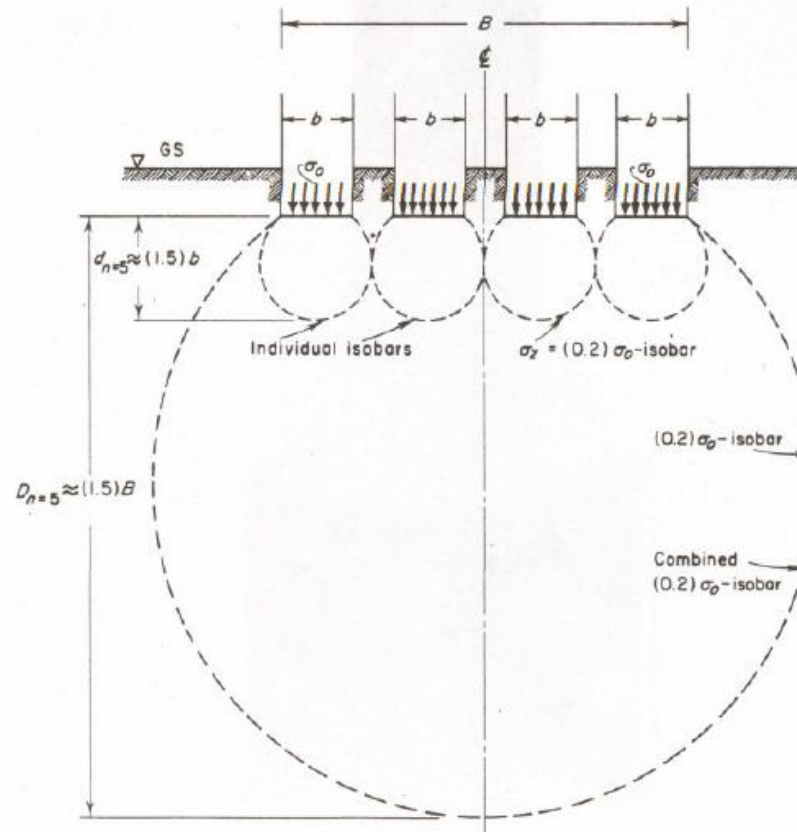
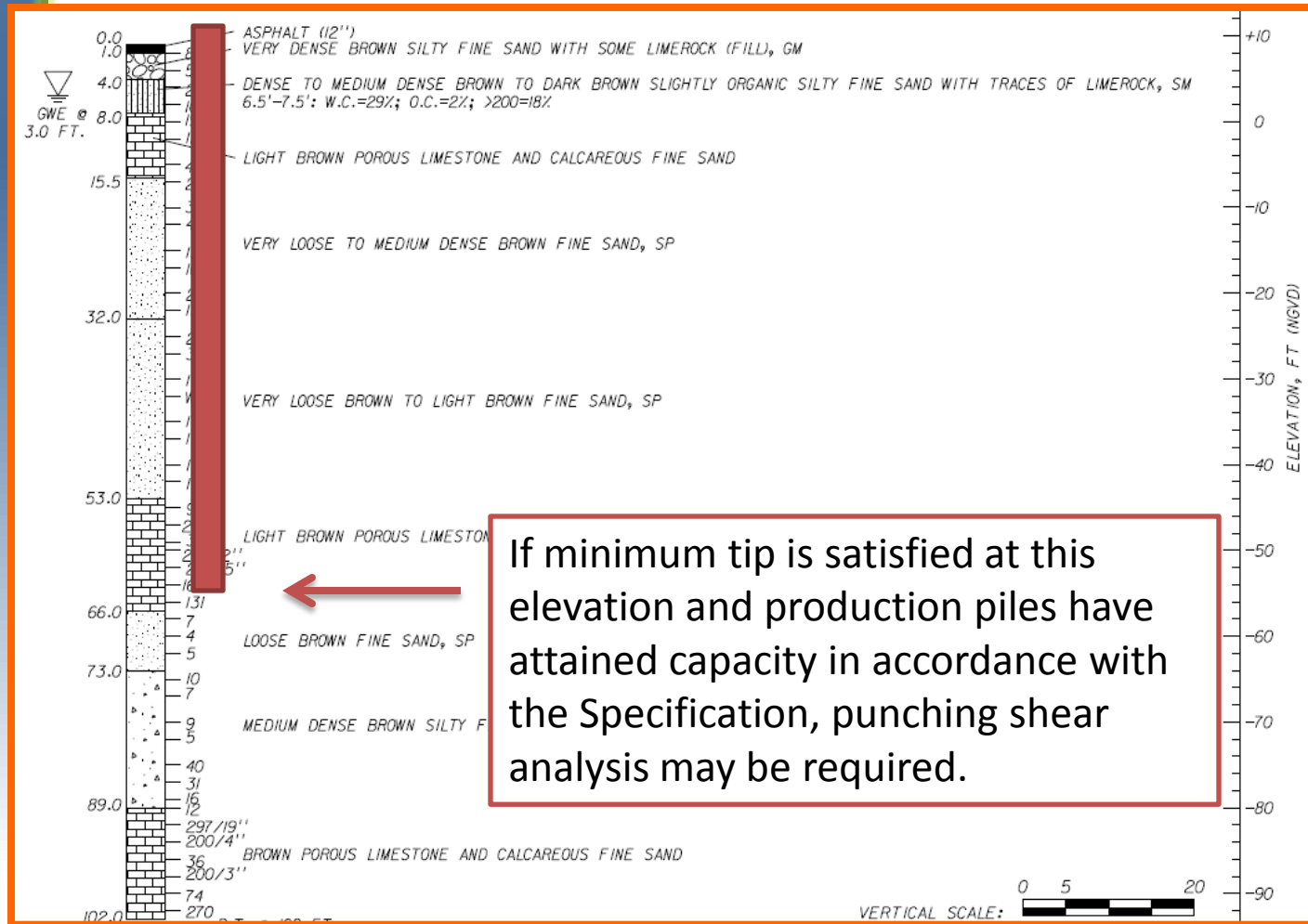


Figure 6. Closely spaced isobars merge into one isobar of the same intensity but reaching far deeper than the individual isobars.

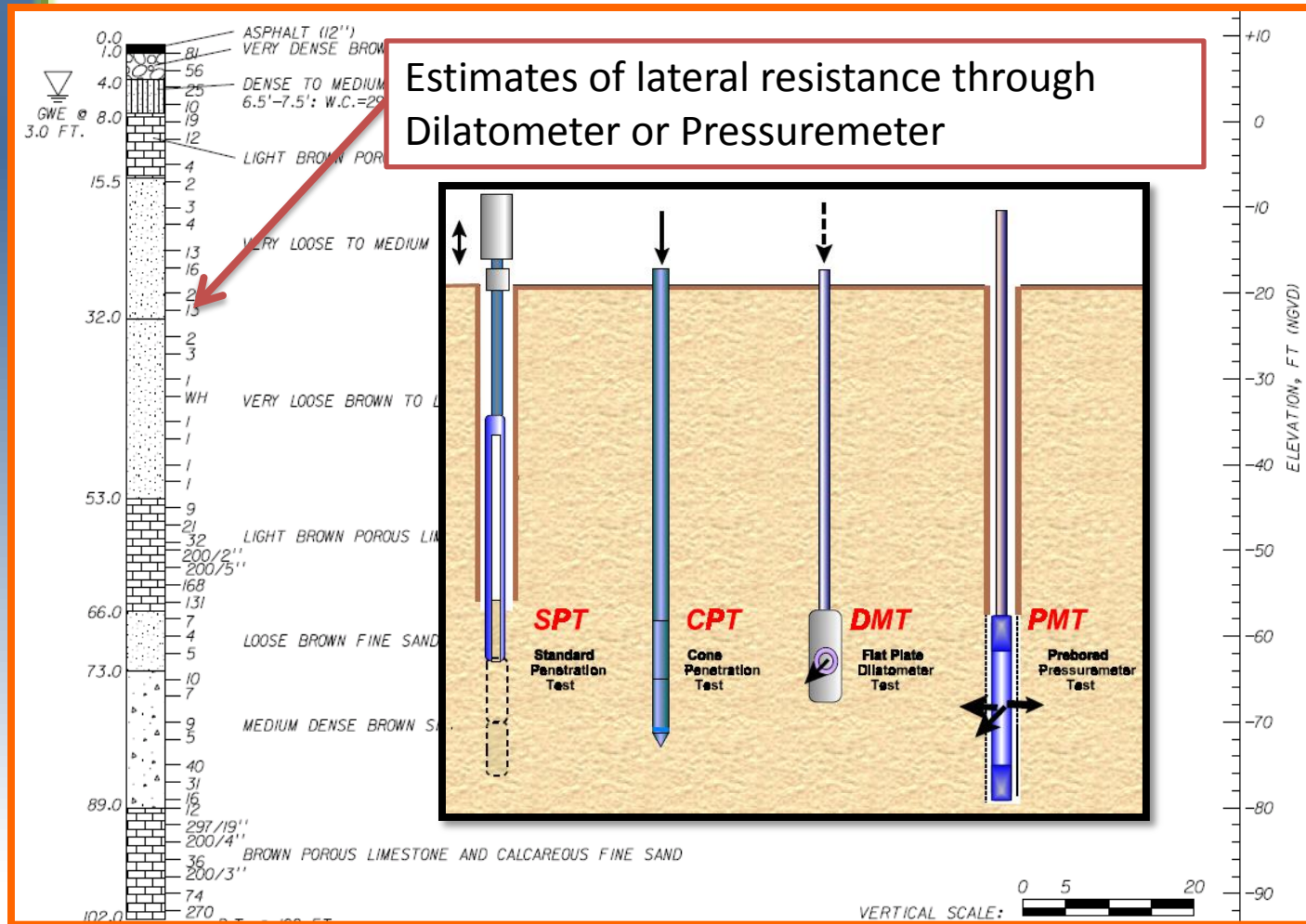


# PUNCHING SHEAR





# PUNCHING SHEAR



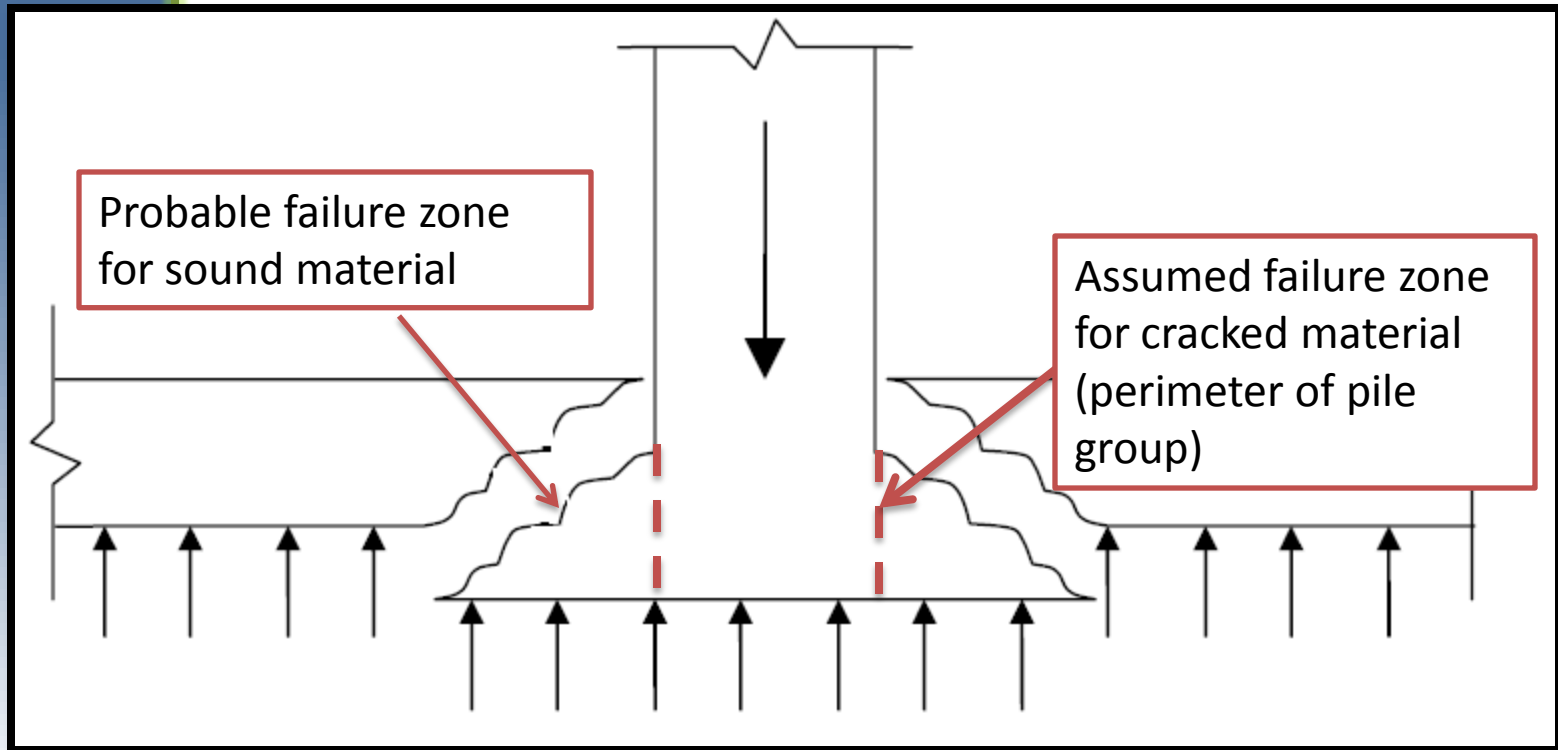


# PUNCHING SHEAR

- Rough estimates of rock (Limestone) shear strength based on SPT blow count
  - $f_{su} = 0.1 * N$  (tsf) where  $f_s \leq 5.0$
  - Laboratory results on rock core samples will provide more accurate estimates
  - $f_{su} = 0.5 \sqrt{q_u} \sqrt{q_t}$  REC

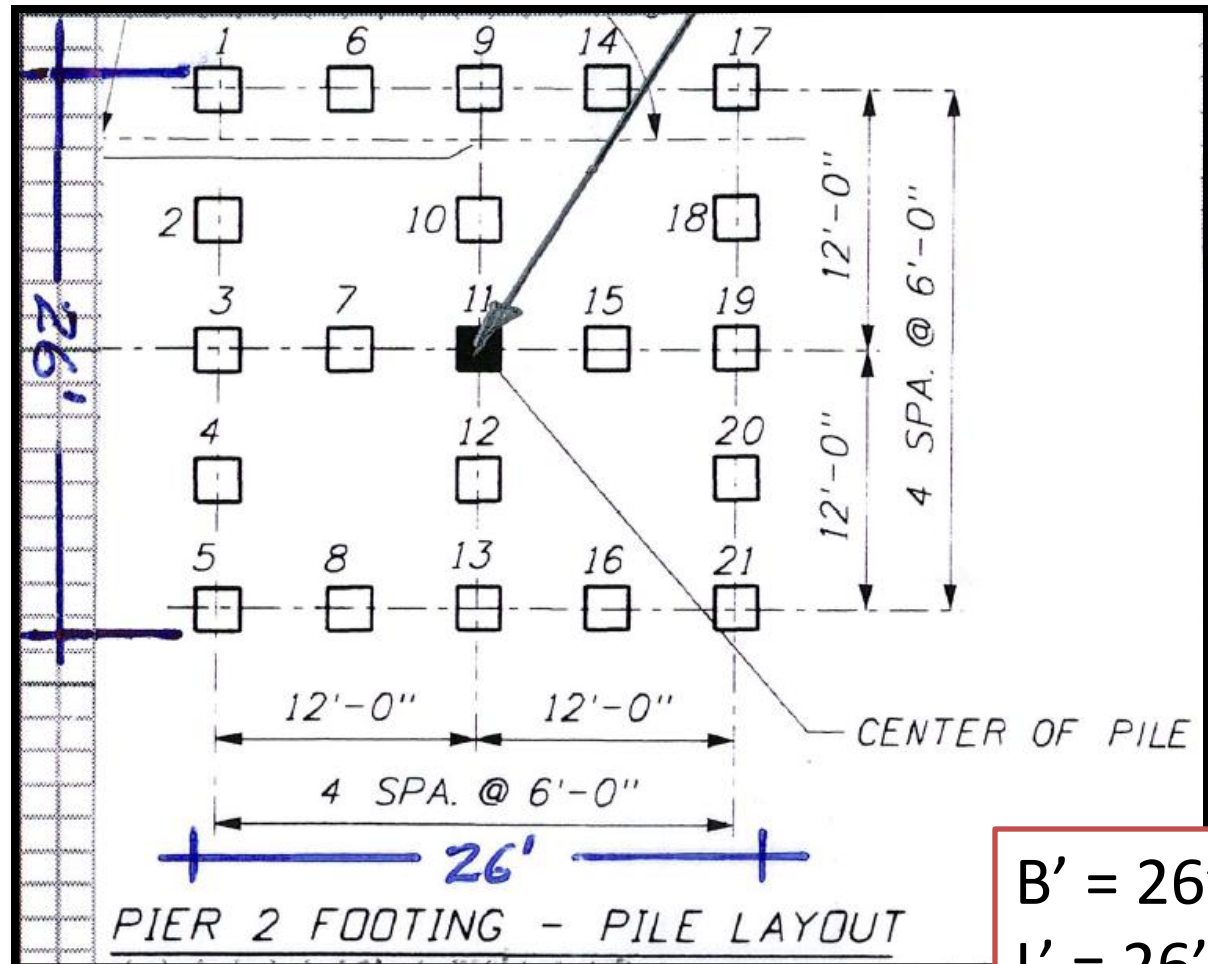


# PUNCHING SHEAR





# PUNCHING SHEAR

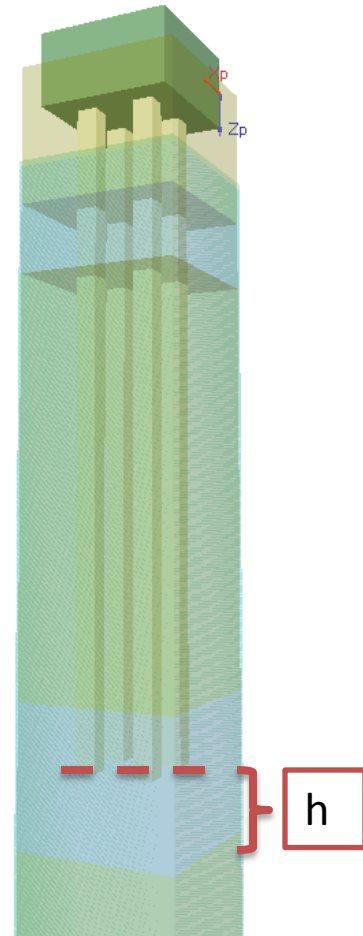


$$B' = 26'$$
$$L' = 26'$$



# PUNCHING SHEAR

- Shear area  
 $= (2 * h * B') + (2 * h * L')$   
 $= 2h(B' + L')$





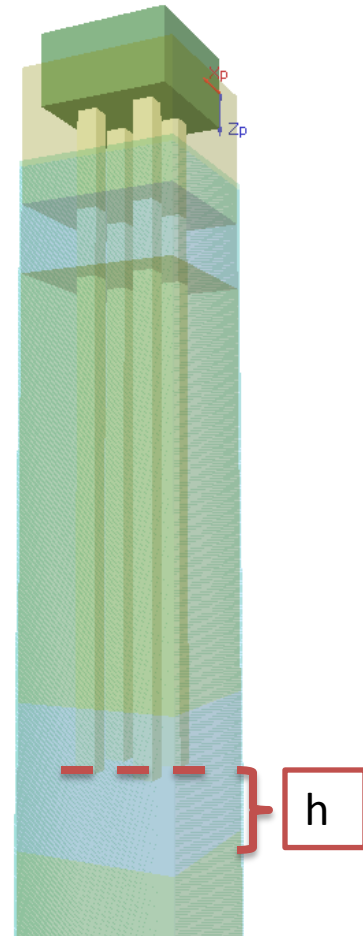
# PUNCHING SHEAR

- Minimum thickness below piles

$$h = Q / [2(B' + L')\phi f_{su}]$$

**Q = Factored load on the group, transferred to pile tip**

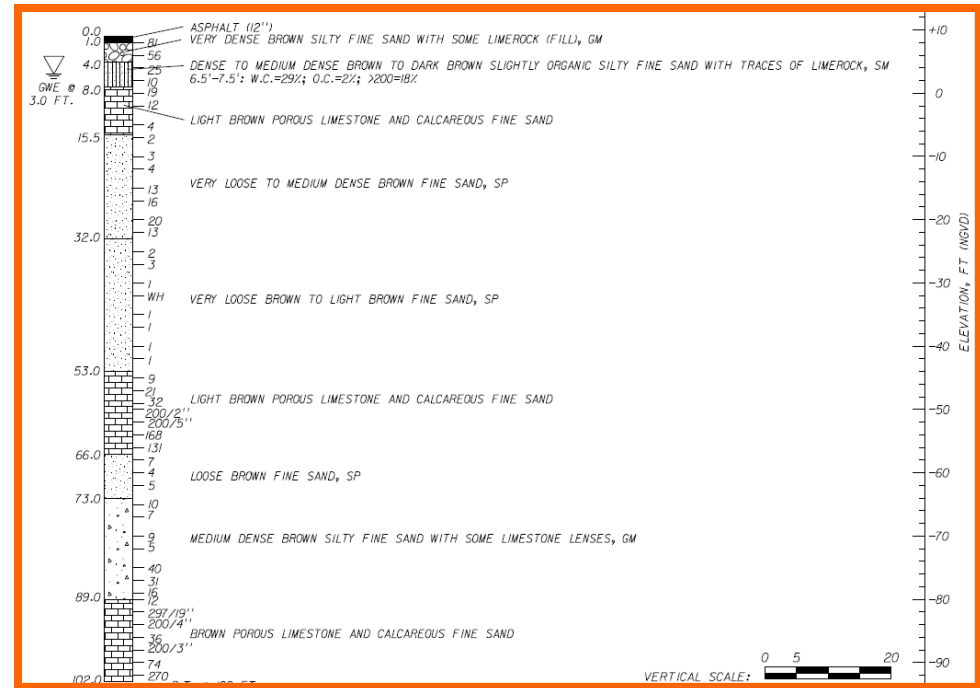
$$\Phi = 0.5 - 0.65$$





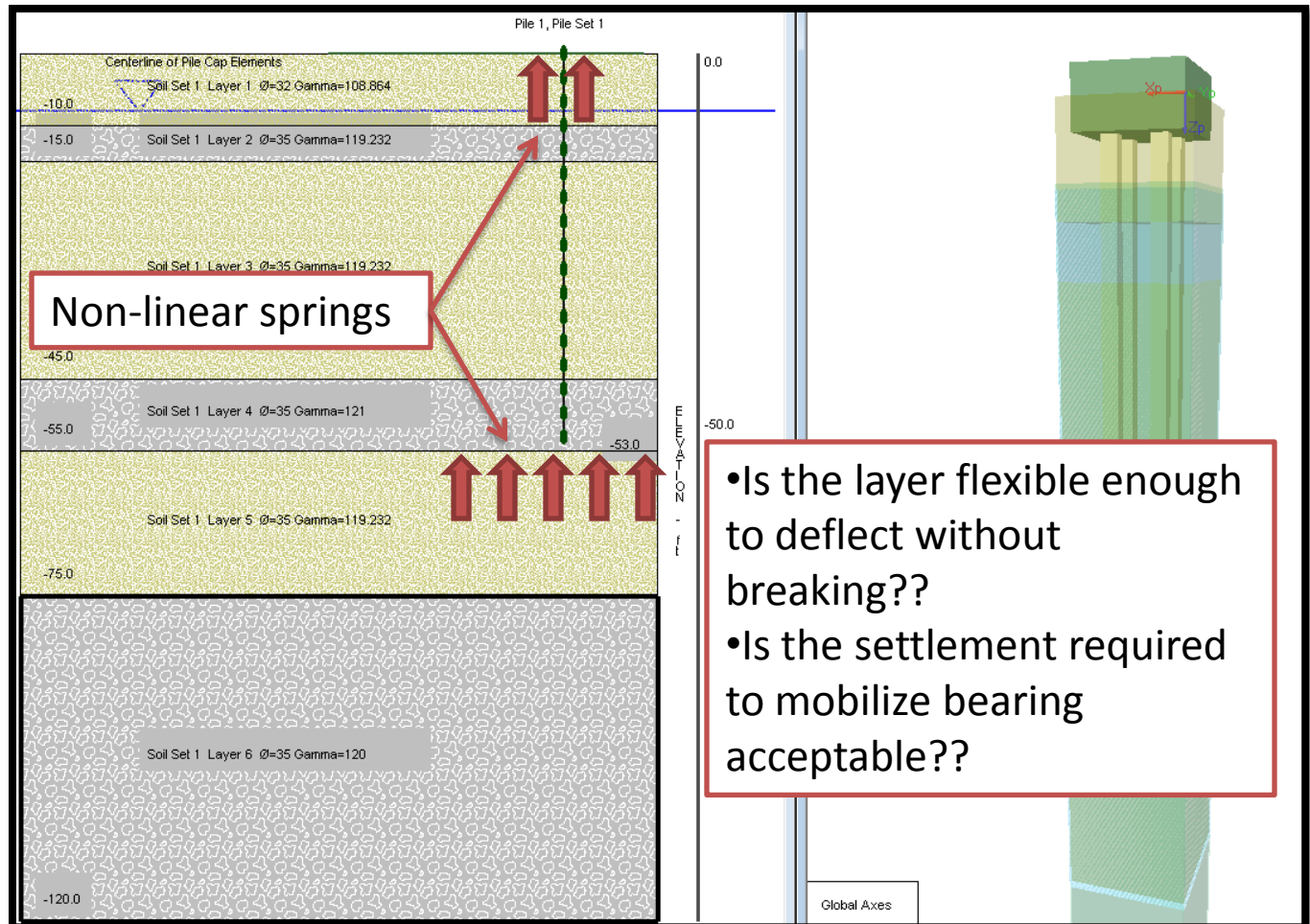
# PUNCHING SHEAR

- Contribution from the soil below the rock?
- Contribution from the cap?
- SETTLEMENT OF ROCK LAYER



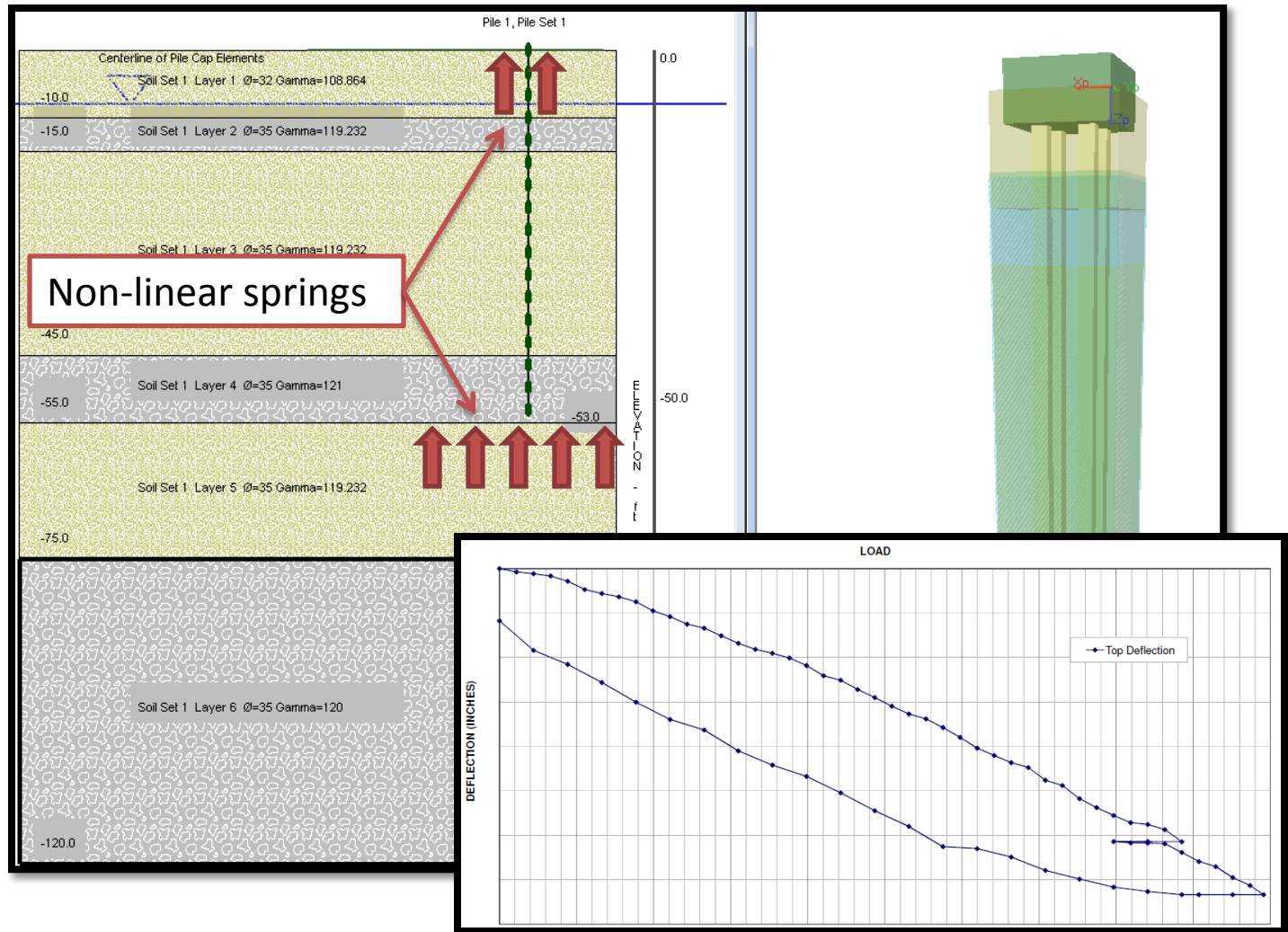


# PUNCHING SHEAR





# PUNCHING SHEAR





# PUNCHING SHEAR - SUMMARY

- “Floating” rock layers - Perform the analysis when piles tip near the bottom of the rock
- Ignore contribution of underlying soil unless a more advanced analysis is available (e.g., finite element) and the anticipated settlement is acceptable



# PUNCHING SHEAR - SUMMARY

- Implement re-strikes to check on excessive rock fracturing due to adjacent pile installation



# DESIGN EXPO 2012

- Questions?
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